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preliminary
engineering report

OLDMAN RIVER DAM

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AGNES



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1986 03 14

File No. 2496-026-223-1.1

Alberta Environment
Oldman River Dam Project Office
Main Floor, Oxbridge Place
9820 - 106 Street
EDMONTON, Alberta
T5K 2J6

Attention: Dr. R. N. McManus, P.Eng.
Project Manager

Dear Sirs:

RE: OLDMAN RIVER DAM
PRELIMINARY ENGINEERING REPORT
AND COST ESTIMATE



We are pleased to submit our preliminary engineering report and cost estimate for the Oldman River Dam flow control project.

The engineering work summarized herein included an extensive site investigation program to establish subsurface design parameters, a study of structure layout alternatives, the development of a construction strategy and schedule, and the preparation of a preliminary project cost estimate supported by estimates prepared by two independent organizations. The study has recommended that the project be constructed with a right bank spillway, two left bank diversion tunnels, and a multi-zoned earth and rockfill dam, at a total cost estimated at \$264,300,000 in 1985 dollars, and can be completed in time to capture the 1991 spring run-off.

The investigations also included an evaluation of the hydro-electric potential of the site, which concluded that a power installation may be economically justified and warrants the installation of certain minimum facilities at a cost of \$1,250,000 to permit the future installation of the hydroelectric power station.

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Alberta Environment
1986 03 14
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Certain assumptions have been made during the development of the project design parameters which will require the review and approval of Alberta Environment prior to proceeding with the final design of the facilities. These include:

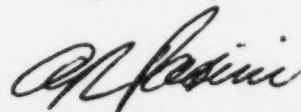
- . the river diversion works will be sized to pass the spring flood with a 1:50 year return period flood.
- . pre-excavation of the spillway plunge pool is not planned and may necessitate some remedial work in the future, if erosion by the spillway jet is greater than anticipated,
- . steep excavated slopes in the spillway approach channel which may require some future maintenance,
- . unreinforced concrete lining in the diversion tunnels downstream of the control valves, which could possibly crack and result in some seepage entering the tunnel,
- . future access to the upstream portion of the tunnels will require a barge and divers to install stoplogs, should it ever be necessary to unwater that portion of the tunnels, and
- . fully automated reservoir operation.

Additional investigations are required in several areas prior to preparing tender documents, including hydraulic model testing of the tunnels and the spillway, a riprap quarry test program, a foundation grouting test program, rock anchor tests, an embankment test fill, additional test pits in granular borrow sites, and additional lab testing of foundation and embankment materials relative to a dynamic design analysis of the stability of the main dam.

We appreciate the opportunity of providing services to you on this phase and look forward to working with you on the final design and construction phases.

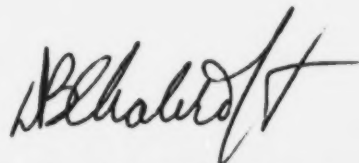
Yours very truly,

UMA Engineering Ltd.



A. R. Pasini, P.Eng.
PRESIDENT

ARP/DBC/pm



D. B. Chalcroft, P.Eng.
PROJECT MANAGER

uma

OLDMAN RIVER DAM
PRELIMINARY ENGINEERING REPORT

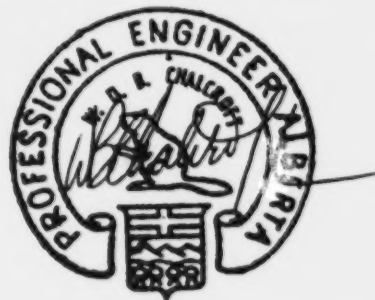
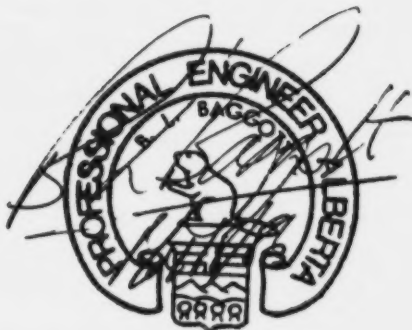
Prepared for:

ALBERTA ENVIRONMENT

Prepared by:

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March, 1986



PERMIT TO PRACTICE UMA ENGINEERING LTD.	
Signature	<i>[Signature]</i>
Date	<i>Mar 14/86</i>
PERMIT NUMBER: P 329	
The Association of Professional Engineers, Geologists and Geophysicists of Alberta	



OLDMAN RIVER DAM REPORT

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- H. Buried Valley Investigation
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SUMMARY

This report records the major results of the preliminary design of the Oldman River Dam project. The preliminary design was carried out during the period January to December 1985.

The Oldman River Dam will provide flow regulation and onstream storage of a dependable supply of water which will permit multi-purpose use and irrigation service area expansion in Southern Alberta. The hydroelectric potential of the site can be economically developed in the future as a secondary benefit. The dam will be constructed at a site located a short distance downstream of the confluence of the Oldman River, Crowsnest River and Castle River. The site is approximately 10 km northeast of Pincher Creek. At this location the Oldman River has eroded a steep sided, narrow valley through bedrock and makes a southerly turn before resuming its northeasterly course through Lethbridge to eventually join the Bow River and form the South Saskatchewan River.

Studies of possible alternative arrangements of the major structures have shown that the recommended project layout, is the optimum with respect to cost, design and performance of the structures. Certain adjustments to the location of the structures have been made to account for the increased spillway capacity requirements and the more detailed knowledge of the subsurface conditions. The spillway will be positioned on the right (west) bank with the chute oriented in the direction of the river downstream of the project site. Both the spillway and the dam have been located to minimize excavation and fill volumes required, and to maximize the use of soil and rock available from the required structure excavations, in the dam fill. The diversion tunnels will be located in the left (east) bank.

During construction, the Oldman River will be diverted through two tunnels 6.5 m in diameter and 900 m long. This will allow the foundation of the dam to be excavated and prepared in the dry. After completing the diversion function, the tunnels will be converted to low level outlets for riparian and multi-purpose releases. Control of reservoir outflow will be by means of a hollow cone valve installed in a concrete plug at the mid-point of each tunnel. Minimum provisions will be made in the east tunnel to accommodate a possible future addition of a hydroelectric powerplant.

The 76 m high dam will be a zoned earth and rockfill embankment with a central core of glacial till and shells of soil and rock obtained from the required spillway excavation, and sand and gravel from the river alluvium upstream of the dam. Internal filters and drains of sand and gravel will control seepage through the dam and its foundation.

The spillway will be 85 m wide and will have seven, 10 m wide gated openings. The gates will be 8.5 m high. The spillway will be capable of passing the entire routed probable maximum flood with the reservoir surcharged 5.6 m above its full supply level. The flood discharges will be conducted back to the river through an 85 m wide chute terminating in a flip bucket energy dissipator.

The principal project data are listed in the following table.

OLDMAN RIVER DAM

PROJECT DATA

RESERVOIR

FuTT Supply Level (FSL)	1,118.6 m
Reservoir Live storage at FSL	490,000 dam ³
Maximum Reservoir Level (PMF surcharged)	1,124.2 m
Reservoir Area at FSL	2,420 ha
Probable Maximum Flood	9,320 m ³ /s

DAM

Dam Crest Elevation	1,125.5 m
Maximum Height	76 m
Freeboard at FSL	6.9 m
Crest Length (including dyke)	3,070 m
Maximum Base Width	615 m
Average Slopes	
- Downstream	3.9:1
- Upstream	4.2:1

SPILLWAY

Crest Elevation	1,111.1 m
Gates	7 @ 10 x 8.5 m
Crest Width	85 m
Spillway Length	352 m
Spillway Discharge	
- at FSL	2,970 m ³ /s
- at PMF (routed)	7,675 m ³ /s

TUNNEL

Diversion Capacity (outflow)	1000 m ³ /s
Outlet Capacity (FSL)	85 m ³ /s
Number and Diameter	2 @ 6.5 m
Average Length	900 m

MAJOR QUANTITIES (m³)

Concrete	161,500
Excavation - soil	1,915,000
- rock	3,482,000
- total	5,397,000
Dam & Dyke - core	1,527,000
- shells	5,375,000
- drains	678,000
- riprap	251,000
- total	7,831,000

The magnitude of the project and river diversion requirements result in an overall construction schedule covering a five year period. Construction will start in March 1986 and the works will be substantially complete by the fall of 1990 to allow the reservoir to fill during 1991. Items on the critical path portion of the schedule include: the diversion works, river diversion, dam foundation excavation, dam fill placement and valve installation. Spillway construction, dyke placement, and spillway gate installation are not on the critical path. Spillway excavation is however linked with construction of the dam.

The estimated capital cost of the project in 1985 dollars is \$264,300,000.

The mean annual precipitation varies between 550 mm in the mountains to the west and 450 mm near the project site. Streamflow data have been recorded in the Oldman River basin since 1909. Although a flood estimated to have a peak flow of $5660 \text{ m}^3/\text{s}$ was reported to have occurred in the Oldman River basin the year before stream gauges were established on the river, the largest floods on record are $2220 \text{ m}^3/\text{s}$ in 1923 and $1560 \text{ m}^3/\text{s}$ in 1975.

The probable maximum flood (PMF) at the project site has been computed to be $9320 \text{ m}^3/\text{s}$, which is substantially larger than the PMF of $4220 \text{ m}^3/\text{s}$ used in previous studies. The primary reason for the difference is that the revised value is based on the 1975 flood hydrograph which had a time to peak of 1.5 days compared with the 4.6 days used previously. The total flood volumes computed for both studies were almost identical.

Inflowing suspended sediment and bed material load will be deposited in the reservoir such that the storage capacity will be reduced on the average by 0.06 percent per year. Of the three major rivers entering the reservoir, it is expected that only the upper Oldman River and Castle River will experience bed aggradation in their respective backwater zones upstream of the reservoir. After 100 years, maximum aggradation amounts of 2 m and 1 m, respectively, can be expected. Channel degradation downstream of the dam due to the trapping of sediment in the reservoir, will ultimately be limited by armouring of the bed. A maximum degradation amount of 0.5 m is predicted in the first 2 km downstream of the project site in about 25 years.

The present ice regime in the vicinity of the project site is very stable and shows no signs of shoving, jamming or packing. This condition will continue after the reservoir is in operation providing that the winter flows are not greater than about twice the present flows and are maintained at a relatively uniform level.

A detailed investigation of the subsurface conditions at the site was carried out together with an inventory of naturally occurring sources of construction materials.

The site is underlain by flat lying, interbedded sequences of claystone, siltstone, and sandstone. Glacial till and glacio-lacustrine silt and clay overlie the bedrock. Alluvial sand and gravel is found in terraces and flood plains along the rivers. The river banks at the site are mantled with colluvium.

The minor terrace deposits of pervious sand and gravel found in a buried valley, which passes to the north of the project site, do not appear to present a possible source of significant leakage from the reservoir. No faults have been found in the vicinity of the project site. Joints pervade all rock types and, although the permeability of the rock is moderately high, it will be possible to grout the foundation by conventional methods.

The use of recently developed coring equipment has resulted in significantly greater core recovery from the rocks encountered than was possible during previous investigations of the project site. In particular, it was possible to obtain relatively undisturbed core of the softer zones which were not recovered previously because they were either washed out or destroyed during drilling. The core-loss zones reported in previous investigations were found to be structural discontinuities in the rock mass or seams containing silt, clay and breccia. An exploratory shaft sunk 50 m deep into the lower part of the right (west) dam abutment and a test tunnel driven into the left (east) river bank has permitted inspection of the rock conditions in situ and the acquisition of large diameter, undisturbed core samples of the seams for laboratory shear box testing.

An analysis of these seams indicates that they could be continuous through the river valley and banks over distances as great as 800 m, and that the shear strength of the material in the seams is close to its residual value. The effect of the presence of these seams on design and layout is to require the dam to have flatter slopes with a consequent lengthening of both the tunnels and the spillway excavation, compared to previous studies.

Adequate fine-grained low permeability material, and sand and gravel have been found for the construction of the dam and for the production of aggregates required for concrete.

A preliminary evaluation of the economic benefits of a possible hydroelectric powerplant installation at the Oldman River Dam showed that it would be financially attractive and worthy of further study. The optimum installed capacity is in the 10 MW to 30 MW range with the latter yielding the maximum net benefits. The value of the benefits is very sensitive to the future real rate of escalation of energy costs. The capital cost to provide the minimum facilities necessary to allow for a future powerplant installation is estimated to be \$1,250,000.

Based on the site investigations and preliminary engineering studies carried out during 1985, it is concluded that the Oldman River Dam can be successfully constructed at the proposed site. Foundation conditions at the site, though complex, can be successfully treated utilizing conventional construction techniques. Given a project start in early 1986, the project can be completed in time to capture the spring run-off in 1991.

SI UNITS OF MEASUREMENT

The following SI units used in the report are presented in this table with the corresponding Imperial System equivalents. The symbols used in the text are listed in alphabetical order.

SYMBOL	SI UNIT		IMPERIAL SYSTEM MEASUREMENT
cm	centimetre	10^{-2} m	0.394 inch
dam	decametre	10 m	32.808 foot
dam ³	cubic decametre	10^3 m^3	0.811 acre foot
g	gram	--	0.002 pound
ha	hectare	10^4 m^2	2.471 acre
kg	kilogram	10^3 g	2.205 pound
kg/m ³	kilogram per cubic metre	--	0.062 pound per cubic foot
kN/m ³	kilonewton per cubic metre	$10^3 \text{ kgm/s}^2/\text{m}^3$	6.365 pound force per cubic foot
km	kilometre	10^3 m	0.621 mile
km ²	square kilometre	10^6 m^2	247.105 acre
kPa	kilopascal	$10^3 \text{ kgm/s}^2/\text{m}$	0.145 pound force per square inch
kt	kilotonne	1000 t	984.206 ton (1)
kW	kilowatt	10^3 W	1.341 horsepower
l	litre	10^{-3} m^3	0.220 gallon
m	metre	--	3.281 foot
mm	millimetre	10^{-3} m	0.039 inch
m/s	metre per second	--	3.281 feet per second
m ³ /s	cubic metre per	--	35.315 cubic feet per second
MN	meganeutron	10^6 kgm/s^2	224,809 pound force
MPa	megapascal	$10^6 \text{ kgm/s}^2/\text{m}$	145.038 pound force per square inch
t	tonne	1000 kg	0.984 ton (1)

Note: (1) Long ton = 2240 pound

SECTION 1
INTRODUCTION

1.0 INTRODUCTION

This report records the major results of the preliminary engineering of the Oldman River Dam project. It includes a description of the investigation of the project site, the layout and preliminary design of the component structures, the construction schedule, an estimate of the capital cost of the project and an evaluation of the hydroelectric power potential.

1.1 AUTHORIZATION

Authorization to proceed with the preliminary engineering phase was received from Alberta Environment on January 9, 1985. The work has been executed by UMA Engineering Ltd. under Alberta Environment Agreement No. 85-0750 dated April 17, 1985.

1.2 SCHEDULE

Work on preliminary engineering was initiated on January 10, 1985, and was essentially completed by November 29, 1985. However, some tasks, included in the preliminary engineering phase, are still underway and will be reported separately. These have been scheduled for completion after the preparation of this report because they relate either, to additional site data that were identified during preliminary design as being essential input to final design or to work

that could not be started until the preliminary design had been completed. These tasks include drillholes located at or close to the final structure locations, test pit data that will be required by contractors while preparing tenders for construction of the works, rock anchor testing, a test tunnel that will provide tenderers with information on underground excavation in the rock conditions existing at the project site, a riprap quarry investigation, a pilot foundation grouting program, and hydraulic models of the spillway and tunnels, that will be used to confirm the hydraulic design of the structures. These additional tasks will be undertaken subsequent to submission of the Preliminary Engineering Report and will be reported separately.

1.3 ORGANIZATION

Preliminary design, associated site investigations and construction planning have been carried out by UMA Engineering Ltd., as the prime consultant, in association with Acres International Limited and nine sub-consultants.

Most of the work was carried out either in the project office located in the UMA building in Edmonton or from a project field office located in the Town of Pincher Creek.

The preliminary engineering was divided into 37 work packages for cost and schedule control purposes and to allow discrete tasks to be identified and assigned to the nine sub-consultants. These were assigned on the basis of each sub-consultant's area of specialization and experience. The work packages assigned to sub-consultants during the preliminary engineering phase were:

- . Angus Butler Engineering (1980) Ltd.
 - electrical design
- . Brown Okamura and Associates Ltd.
 - surveying
- . CSC Project Management Services Ltd.
 - independent capital cost estimate
- . The Foundation Company of Canada Limited
 - independent capital cost estimate
- . Geo-physi-con Co. Ltd.
 - geophysical surveys
- . Golder Associates (Western Canada) Ltd.
 - tunnel exploration
 - preliminary tunnel design
 - construction materials inventory
- . Hydrocon Engineering (Continental) Ltd.
 - flood frequency analyses
- . Northwest Hydraulic Consultants Ltd.
 - sedimentation and river engineering
 - fuse plug spillway scour study
 - hydraulic models of the tunnels and spillway
- . Thurber Consultants Ltd.
 - dam and spillway foundation exploration
 - buried valley investigation

The Hydrology Branch of Alberta Environment carried out the probable maximum flood analysis for the project.

A technical review committee consisting of four specialist consultants reviewed the preliminary engineering at periodic intervals during its progress.

1.4

PROJECT BACKGROUND

The Oldman River Dam will provide flow regulation and onstream storage of a dependable supply of water for multi-purpose uses. One of the benefits will be to support

expansion of the irrigation systems in Southern Alberta. Other benefits will include more efficient and effective management of the water resources of the South Saskatchewan River basin, the maintenance of minimum river flows to meet the requirements of the inter-provincial apportionment agreement, assured water supply for domestic, municipal and industrial needs and a potential for hydroelectric power generation.

The water use and water supply capacity of the Oldman River basin have been studied at various times since 1958; initially by the Prairie Farm Rehabilitation Administration (PFRA) and more recently by Alberta Environment. The studies were undertaken as part of the province-wide appraisal of its water resources and as input to the planning for management of these resources on a river basin basis. The possibility of regulating the flow of the Oldman River and improving the water supply in the Oldman River basin were investigated during the previous studies.

On August 9, 1984, the Government of Alberta announced the siting and its decision to proceed with construction of the Oldman River Dam.

The project consultants were selected in January 1985, and preliminary design and detailed investigations of the project site commenced.

The design has made full use of the reports and other data and information collected during the previous studies of the project. These are listed in the bibliography of this report.

1.5 PROJECT LOCATION

The Oldman River Dam site is located in Sec 17, Twp 7, Rge 29, W4M, approximately 10 km northeast of Pincher Creek, as shown on Plate 1. The site is a short distance downstream of the confluence of the upper Oldman River, Crowsnest River and Castle River.

1.6 SITE DESCRIPTION

At the project site the Oldman River valley is a relatively narrow channel eroded with steep banks through bedrock. The flat valley bottom is approximately 250 m wide and is at approximate elevation 1,055 m. The 60 m wide river flows against the left valley slope. On the average, the left valley rises at 38° and the right valley rises at 27° to the flat uplands at elevation 1,120 m. The valley is approximately 400 m wide at the upland level. The reservoir will extend upstream some 24 km and will affect an area of 2,420 ha. The project site is shown on Plate 2.

1.7 ACCESS

The project site is accessible from Highway 3. The east-west CP Rail line through the Crowsnest Pass passes to the south of the project site through Pincher Station. There is a 2000 m long paved airport approximately 4 km from Pincher Creek. These are shown on Plate 1.



SECTION 2

HYDROLOGY

2.1 INTRODUCTION

Hydrologic studies have been carried out to establish design floods for the component structures of the Oldman River Dam project, and to provide flow sequences to the power benefits study. The hydrologic information contained in previous studies by PFRA, Alberta Environment, and other agencies, was reviewed and updated.

Three separate studies were undertaken:

- a flood frequency analysis to predict the flood peaks and flood volumes expected to occur during various return period events,
- a probable maximum flood (PMF) study to estimate the largest flood event that could possibly occur, and
- the development of reservoir operation scenarios to assess the power benefits that might accrue to the project.

The latter study is described in Section 14.

Automatic stream gauges and staff gauges were established along the reach of the river in the immediate vicinity of the project site. Readings were taken during the 1985 summer period and will be continued.

2.2 OLDMAN RIVER BASIN

The Oldman River originates at a small alpine lake in the Rocky Mountains about 100 km southwest of Calgary as shown on

Plate 3. It flows in a southeasterly direction to its confluence with the Crowsnest River and Castle River near Cowley and then swings northeast past Fort MacLeod. Some 15 km below the Willow Creek confluence, the river turns southeastward again and is joined by its other major tributaries, the Belly River and St. Mary River. It then resumes a northeasterly course through Lethbridge and eventually joins the Bow River, to form the South Saskatchewan River. The total drainage area of the Oldman River basin is 26,000 km², with approximately 23,900 km² located in Alberta.

2.2.1 Physiography

The physiography of the river basin can be simply divided into the mountain region and the foothills region.

The mountain region covers about a 20 km wide belt between the continental divide to the west and the foothills to the east. The local relief is extreme. The valleys of the larger streams generally lie below elevation 1500 m. The surrounding peaks reach elevations between 2400 m and 3100 m. The timber line lies at about elevation 2100 m. Most of the region below that level is covered with a heavy forest where spruce and pine predominate.

The foothills region along the eastern flank of the mountains consists of north-south trending ridges separated by steep sided valleys. To the east the relief gradually smooths out, as the foothills become lower and less distinct, and finally merge into the high plains at about elevation 1220 m. The vegetation in the foothills region varies from heavy forest to rolling short grassland.

2.2.2 Climate

Mean annual precipitation in the Oldman River basin generally decreases from west to east. The values, reported by Alberta Environment, for the mountain region are in the order of 600 mm and for the foothills region vary from about 550 mm along the eastern flanks of the mountains to about 450 mm in the general vicinity of the project site.

About one half of the annual precipitation in the mountain region occurs as snowfall. Snowfall is less significant in the foothills region, where it constitutes between one third and one half of the annual precipitation.

Normal summer precipitation in the Oldman River basin occurs when low pressure weather systems cross the continental divide from the west. However, a thrust of tropical maritime air from the south can also be forced upward against the foothills and eastern slopes of the mountain, releasing heavy precipitation. Such storms can be particularly intense in the most southerly portion of the basin where the more severe topographic relief provides greater uplift to the tropical air masses.

2.2.3 Surface Water Runoff

Streamflow records have been collected on the Oldman River since as early as 1909 as shown on Plate 3. Records for the Water Survey of Canada (WSC) gauge (05AB013) located near Brocket for the period from 1966 have been extended by Alberta Environment to cover the period of record for the basin from 1910. The mean monthly flows for the period are listed in Table 2-1. These records indicate that the flow in

TABLE 2-1
MEAN MONTHLY STREAMFLOW RECORD

(m³/s)

YEAR	JAN.	FEB.	MAR.	APR.	MAY	JUN.	JUL.	AUG.	SEP.	OCT.	NOV.	DEC.
1912	9.4	8.8	15.3	38.2	92.9	78.7	62.7	28.5	16.0	15.1	16.2	8.9
1913	7.6	2.4	3.7	52.7	119.8	161.0	51.1	32.2	20.4	19.8	15.7	11.1
1914	7.0	7.0	9.0	35.0	106.0	82.0	33.6	19.0	17.0	30.0	21.0	11.0
1915	9.0	7.4	11.0	37.0	149.0	172.0	88.8	56.0	27.0	28.0	15.0	13.0
1916	8.0	17.0	23.0	39.0	101.0	273.0	111.0	29.0	24.0	16.0	13.0	13.0
1917	8.0	8.0	8.0	35.0	231.0	254.0	68.0	20.0	14.0	11.0	9.0	7.8
1918	12.0	10.0	13.0	31.0	87.0	114.0	31.0	17.0	17.0	15.0	9.0	7.7
1919	7.0	6.0	5.0	27.0	116.0	80.0	25.0	12.0	9.0	15.0	9.0	7.7
1920	7.0	8.0	15.0	32.0	137.0	144.0	82.0	22.0	14.0	15.0	10.0	7.7
1921	7.0	8.0	12.0	34.0	127.0	137.0	45.0	17.0	12.0	15.0	10.0	7.7
1922	7.0	6.0	6.0	25.0	129.0	138.0	43.0	16.0	10.0	15.0	10.0	7.7
1923	7.0	5.0	7.0	20.0	98.0	378.8	73.0	34.0	16.0	15.0	10.0	7.7
1924	7.0	9.0	19.0	19.0	95.0	132.0	38.0	27.0	14.0	15.0	10.0	7.7
1925	11.0	9.6	15.4	66.0	148.3	114.8	38.8	16.3	17.2	15.0	10.0	7.7
1926	8.0	7.1	30.5	42.0	41.6	46.3	26.8	11.5	35.5	33.9	12.0	12.0
1927	8.0	12.0	12.0	42.0	200.0	328.0	101.0	11.0	44.0	43.0	12.0	12.0
1928	18.0	12.0	24.0	39.0	175.0	152.0	135.0	33.0	23.0	22.0	12.0	12.0
1929	8.0	12.0	13.0	22.0	139.0	177.0	37.0	14.0	22.0	22.0	12.0	12.0
1930	6.0	11.0	59.0	10.0	126.0	118.0	39.0	14.0	22.0	22.0	12.0	12.0
1931	6.0	11.0	5.0	10.0	63.0	48.0	17.0	11.0	22.0	22.0	12.0	12.0
1932	8.0	8.0	8.0	28.0	135.0	159.0	37.0	11.0	22.0	22.0	12.0	12.0
1933	7.0	7.0	7.0	24.0	114.0	135.0	55.0	11.0	22.0	22.0	12.0	12.0
1934	15.0	15.0	15.0	88.0	152.0	130.0	39.0	11.0	22.0	22.0	12.0	12.0
1935	11.0	14.0	14.0	27.0	83.0	94.0	38.0	11.0	22.0	22.0	12.0	12.0
1936	5.0	13.0	13.0	15.0	88.0	67.0	14.0	11.0	22.0	22.0	12.0	12.0
1937	4.0	13.0	13.0	15.0	74.0	152.0	34.0	11.0	22.0	22.0	12.0	12.0
1938	9.0	16.0	16.0	36.0	173.0	141.0	50.0	11.0	22.0	22.0	12.0	12.0
1939	7.0	16.0	9.9	26.6	82.2	84.7	39.9	11.0	22.0	22.0	12.0	12.0
1940	7.0	10.0	10.0	24.0	98.0	56.0	30.0	11.0	22.0	22.0	12.0	12.0
1941	8.0	8.0	8.0	20.0	51.1	44.0	27.7	11.0	22.0	22.0	12.0	12.0
1942	9.0	7.0	9.0	30.0	220.0	292.0	96.0	11.0	22.0	22.0	12.0	12.0
1943	8.0	14.0	14.0	83.0	125.0	166.0	72.0	11.0	22.0	22.0	12.0	12.0
1944	5.0	5.0	5.0	10.0	38.0	48.0	22.5	11.0	22.0	22.0	12.0	12.0
1945	6.0	6.0	7.0	8.0	103.0	181.0	55.0	11.0	22.0	22.0	12.0	12.0
1946	7.0	13.0	33.0	35.5	113.0	131.0	43.0	11.0	22.0	22.0	12.0	12.0
1947	9.0	13.0	33.0	35.5	113.0	131.0	43.0	11.0	22.0	22.0	12.0	12.0
1948	10.0	10.0	10.0	10.0	298.0	437.8	70.0	11.0	22.0	22.0	12.0	12.0
1949	7.0	11.0	11.0	25.0	115.0	88.0	30.0	11.0	22.0	22.0	12.0	12.0
1950	7.0	11.0	11.0	25.0	115.0	88.0	30.0	11.0	22.0	22.0	12.0	12.0
1951	9.0	10.0	19.0	94.0	127.0	243.0	148.0	11.0	22.0	22.0	12.0	12.0
1952	10.0	10.0	19.0	94.0	127.0	243.0	148.0	11.0	22.0	22.0	12.0	12.0
1953	8.0	10.0	12.0	41.0	176.0	4.0	35.0	11.0	22.0	22.0	12.0	12.0
1954	8.0	10.0	12.0	41.0	176.0	4.0	35.0	11.0	22.0	22.0	12.0	12.0
1955	8.0	10.0	12.0	41.0	176.0	4.0	35.0	11.0	22.0	22.0	12.0	12.0
1956	8.0	10.0	12.0	41.0	176.0	4.0	35.0	11.0	22.0	22.0	12.0	12.0
1957	8.0	10.0	12.0	41.0	176.0	4.0	35.0	11.0	22.0	22.0	12.0	12.0
1958	8.0	10.0	12.0	41.0	176.0	4.0	35.0	11.0	22.0	22.0	12.0	12.0
1959	9.0	13.0	13.0	39.0	160.0	194.0	67.0	11.0	22.0	22.0	12.0	12.0
1960	10.0	20.0	20.0	43.0	104.0	111.0	37.0	11.0	22.0	22.0	12.0	12.0
1961	6.0	11.0	15.0	14.0	139.0	147.0	32.0	11.0	22.0	22.0	12.0	12.0
1962	7.0	11.0	15.0	54.0	89.0	98.0	32.0	11.0	22.0	22.0	12.0	12.0
1963	7.0	9.6	7.4	11.9	68.6	139.6	106.7	18.4	11.0	22.0	12.0	12.0
1964	7.0	6.1	6.1	16.5	175.5	260.3	62.7	17.5	11.0	22.0	12.0	12.0
1965	8.5	9.6	12.1	33.6	100.8	218.6	85.1	29.4	11.0	22.0	12.0	12.0
1966	9.0	9.6	14.0	42.0	129.0	141.0	53.0	22.0	11.0	22.0	12.0	12.0
1967	8.0	12.0	17.0	17.0	210.0	305.0	75.0	21.0	11.0	22.0	12.0	12.0
1968	13.0	11.0	11.6	15.9	101.6	132.8	50.4	22.7	11.0	22.0	12.0	12.0
1969	13.0	11.0	14.0	73.0	131.0	167.0	83.0	19.0	11.0	22.0	12.0	12.0
1970	13.0	11.0	11.5	11.5	127.9	155.7	36.8	13.7	11.0	22.0	12.0	12.0
1971	13.0	11.0	11.5	11.5	127.9	155.7	36.8	13.7	11.0	22.0	12.0	12.0
1972	13.0	11.0	11.5	11.5	127.9	155.7	36.8	13.7	11.0	22.0	12.0	12.0
1973	11.2	10.4	11.7	19.3	94.5	92.5	29.8	13.6	11.0	22.0	12.0	12.0
1974	9.7	9.5	14.8	48.3	166.7	244.9	71.5	30.9	11.0	22.0	12.0	12.0
1975	6.9	6.1	13.3	21.8	127.9	275.0	99.9	32.8	11.0	22.0	12.0	12.0
1976	11.2	11.2	13.3	39.7	143.2	79.7	48.8	90.9	11.0	22.0	12.0	12.0
1977	11.2	11.2	13.3	39.7	143.2	79.7	48.8	90.9	11.0	22.0	12.0	12.0
1978	11.2	11.2	13.3	39.7	143.2	79.7	48.8	90.9	11.0	22.0	12.0	12.0
1979	6.9	6.6	16.3	25.3	131.1	101.2	30.3	15.3	11.0	22.0	12.0	12.0
1980	4.2	5.6	6.8	47.9	123.6	103.3	26.6	15.7	11.0	22.0	12.0	12.0
1981	14.6	13.1	12.9	33.0	213.9	157.3	56.6	26.8	11.0	22.0	12.0	12.0
1982	3.9	5.2	7.5	15.7	85.3	134.5	51.2	15.4	11.0	22.0	12.0	12.0

Reference: South Saskatchewan River Basin Planning Model.

the river, like most other mountain-fed streams in Alberta, is highly variable. It begins to rise in late March or early April, as runoff from snowmelt at lower elevations begins to reach the stream. The rise continues through May and June as warmer temperatures gradually invade the snow fields in the mountains. Heavy rains coincident with the mountain snowmelt period may create flooding in some years.

The recession from high summer flows usually begins in July and continues throughout the fall and winter. However thunderstorms with intense precipitation can occasionally produce moderately high discharges during the period from July through to early October.

There have been several major flood events in the Oldman River basin since the collection of records began in 1909. The highest flows ever recorded in this particular reach of the river were in June 1923, when the peak discharge was reported to be $2220 \text{ m}^3/\text{s}$. The second highest and most recent flood of $1560 \text{ m}^3/\text{s}$ was recorded in June 1975. Records of the latter flood are well documented.

Storms of the cold-low type are also known to have produced high flows throughout much of southern Alberta in 1897, 1902, and 1908. It is reported that the Oldman River at Lethbridge (WSC gauge 05AD007) had a peak discharge of about $5660 \text{ m}^3/\text{s}$ during the 1908 flood. However there is very little other information available concerning these storms and thus their relative magnitude in terms of peak discharges in the vicinity of the project site has never been reliably determined.

2.3 FLOOD FREQUENCY ANALYSIS

2.3.1 Data Sources

Records for the WSC gauge, (05AB013) located near Brocket, some 5 km downstream of the project site, were used as the basis for the flood frequency analysis. This is considered reasonable as there are no regulating structures on the Oldman River between the gauge and the project site and the intervening drainage area is less than 5 percent of the total drainage area upstream of the gauge. Continuous stream flow data were collected at this location between 1966 and 1984 as shown on Plate 3.

2.3.2 Flood Record Extension

The flood records at the Brocket gauge were extended to cover a 73 year period to improve the results of the frequency analysis. The records of daily discharge for selected gauging stations above the project site were first combined. The annual maximum daily discharges and annual maximum 3, 5 and 7 day flood volumes obtained from these records were then utilized, in conjunction with similar data collected at the gauging stations near Fort MacLeod (05AB007) and Monarch (05AD019), to estimate the missing data for the periods from 1910 to 1931 and from 1934 to 1965. The annual maxima for the years 1932 and 1933 could not be estimated due to insufficient data.

2.3.3 Selection of Frequency Distribution

Five commonly used frequency distributions were applied to the annual flood data and the results were evaluated, using

as a basis the expected probability of selected events being equalled and exceeded in the 73 years of record. From this evaluation, the Pearson Type III distribution was determined to be the most reliable distribution for estimating the annual flood probabilities. Some of the other distributions gave comparable results.

2.3.4 Results

The annual flood estimates, all of which apply to the May to June runoff period, are summarized in Table 2-2. For construction planning purposes, a frequency analysis was also undertaken for the individual months of April, July, August, September and October using extended flood records from the gauging station near Brocket. The results of this analysis, which was also undertaken utilizing a Pearson Type III frequency distribution, are summarized in Table 2-3. The flood hydrographs were derived using the results presented in Table 2-3 and a basic hydrograph shape which was patterned after the June 1975 flood of record for the gauge near Brocket. These are shown on Plate 4. The 1975 flood was found to be close to the 50 year event.

The major findings of the flood frequency analysis are:

- The peak flows for return periods of up to 100 years generally agree with those derived for the most recent studies of the project.

TABLE 2.2
ANNUAL FLOOD FREQUENCY ESTIMATES
OLDMAN RIVER NEAR BROCKET

ANNUAL PROBABILITY OF EXCEEDENCE	RECURRENCE INTERVAL	MAXIMUM DAILY DISCHARGE	MAXIMUM INSTANTANEOUS DISCHARGE	FLOOD VOLUME 3 DAY	5 DAY	7 DAY
(%)	(years)	(m ³ /s)	(m ³ /s)	(m ³ x 10 ⁶)		
50	2	240	300	60	95	125
20	5	480	600	105	160	205
10	10	695	870	140	205	260
4	25	1,005	1,255	190	265	330
2	50	1,250	1,560	230	310	380
1	100	1,500	1,880	270	355	430
0.2	500	2,110	2,640	365	460	545
0.1	1,000	2,375	2,970	405	500	590
0.01	10,000	3,270	4,090	540	650	750

NOTE:

1. Maximum daily discharge and flood volumes were determined from a Pearson Type III frequency distribution.
2. Maximum instantaneous discharges were determined by applying a multiplying factor of 1.25 to the corresponding maximum daily discharges.

TABLE 2.3
MONTHLY FLOOD FREQUENCY ESTIMATES
OLDMAN RIVER NEAR BROCKET

ANNUAL PROBABILITY OF EXCEEDENCE	RECURRENCE INTERVAL	MAXIMUM INSTANTANEOUS DISCHARGE					
		APRIL	MAY & JUNE	JULY	AUGUST	SEPTEMBER	OCTOBER
(%)	(years)	(m ³ /s)					
50	2	90	300	110	30	15	15
20	5	145	600	190	65	45	40
10	10	180	870	255	105	85	65
4	25	225	1255	335	165	155	105
2	50	255	1560	400	220	220	135
1	100	285	1880	460	275	275	170
0.2	500	350	2640	610	425	435	245
0.1	1,000	375	2970	675	465	510	280
0.01	10,000	465	4090	880	670	755	395

NOTE:

1. Maximum instantaneous discharges were determined by applying a multiplying factor of 1.25 to the corresponding maximum daily discharges, determined from a Pearson Type III frequency distribution.

- The peak flows for rare events such as the 1 in 500 year event are within 10 percent of the estimates of earlier studies. The 1 in 10,000 year maximum instantaneous flood peak is estimated at $4,090 \text{ m}^3/\text{s}$ which is of the same order of magnitude as the PMF that was used as a basis for spillway design in the previous studies. Details of the flood frequency analysis are included in Appendix A.

2.4 PROJECT DESIGN FLOOD

The project design flood recommended in the Alberta Water Resources Act, Dam and Canal Safety Regulation Guidelines is the probable maximum flood for a project of the size of the Oldman River Dam project.

2.5 PROBABLE MAXIMUM FLOOD

2.5.1 General

The probable maximum flood (PMF) is a hypothetical flood at a selected location on a given stream whose magnitude is such that there is virtually no chance of its being exceeded. It is a flood which would have a return period approaching infinity and a probability of occurrence in any particular year approaching zero. The PMF for a particular project is derived from the probable maximum precipitation (PMP) and, where applicable, snowmelt, coupled with the worst flood-producing catchment conditions that can be realistically expected under the prevailing meteorological conditions. The PMP is the greatest depth of precipitation meteorologically possible for a given duration at a particular time of the year. The rationale and details for estimating the PMF are presented in Appendix B. The major findings of this study are described below.

2.5.2 Probable Maximum Precipitation

The PMP estimate was computed using the orographic separation procedure of the World Meteorological Organization. It was based on maximized conditions for a regional storm over the entire $4,400 \text{ km}^2$ of the drainage basin upstream of the project site. The storms experienced in southwest Alberta in 1964 and 1975, with prevailing winds from the northeast direction, were considered as a basis for the analysis. Both orographic and convergence precipitation producing mechanisms were considered. The average 48-hour PMP for the watershed upstream of the Oldman River Dam was computed to be 336 mm. This value compares favourably with the PMP developed by similar studies in the region.

2.5.3 Probable Maximum Flood

The US Army Corps of Engineers rainfall-runoff model identified as the "Streamflow Synthesis and Reservoir Regulation" (SSARR) model, which has been previously calibrated by Alberta Environment for the 1975 Oldman River flood conditions, was adopted for routing the PMP to produce the PMF at the project site. The resulting hydrograph has a peak discharge of $9,320 \text{ m}^3/\text{s}$ and is shown on Plate 4.

The PMF peak discharge is substantially larger than the PMF magnitude of $4,220 \text{ m}^3/\text{s}$ estimated for the previous studies. Virtually all of the difference in the two computed values of peak discharge can be attributed to the difference in the time-to-peak of the PMF hydrographs; the revised time being 1.5 days based on the 1975 flood experience compared with 4.6

days assumed in the previous estimate. A review of the sensitivity of the major parameters and procedures adopted in deriving the revised PMF indicated that the PMF peak discharge computed would not vary by more than 10 percent.

2.6 RESERVOIR OPERATION

2.6.1 Low Level Releases

The reservoir will be operated to meet the multi-purpose needs of the users downstream. As an example of how the reservoir could be operated, Table 2-4 shows the monthly reservoir releases that were used for the Base Case in the South Saskatchewan River Basin Planning Study.

TABLE 2-4
MONTHLY RESERVOIR RELEASE
SOUTH SASKATCHEWAN RIVER BASIN PLANNING MODEL

YEAR	BASE CASE FLOWS (m ³ /s)											
	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
1928	23.5	17.5	36.7	49.4	180.8	161.0	140.1	34.9	25.1	38.5	19.6	13.0
1929	8.3	7.5	20.8	22.4	127.1	195.1	36.5	22.0	14.4	40.6	0.1	0.0
1930	0.0	0.0	9.3	65.7	121.5	127.1	39.8	14.9	13.3	16.1	0.2	6.2
1931	6.9	5.8	6.9	11.9	54.3	52.4	15.8	15.3	80.2	41.0	0.0	0.0
1932	0.0	6.5	0.0	1.2	95.4	164.2	39.2	19.0	10.5	10.2	12.8	11.0
1933	7.1	4.8	7.0	22.3	113.9	139.8	56.6	30.9	15.1	23.5	22.2	21.4
1934	16.9	19.9	26.3	111.8	162.8	124.8	32.8	15.7	24.3	30.4	0.0	8.0
1935	14.2	18.5	12.3	30.3	92.1	108.3	40.2	16.4	11.6	7.5	2.5	6.5
1936	5.6	4.1	12.4	39.0	92.7	68.6	19.6	21.9	39.6	33.7	0.0	4.3
1937	2.7	0.7	0.0	1.7	82.8	288.6	38.7	15.0	9.7	12.9	9.1	0.0
1938	0.4	0.6	0.0	2.5	202.8	83.8	33.7	21.3	65.9	7.5	0.1	0.0
1939	0.1	0.0	0.0	23.9	72.4	101.9	34.0	14.8	11.7	20.2	5.7	11.6
1940	7.1	6.8	14.8	29.1	100.3	55.9	22.4	14.0	22.5	27.1	17.1	11.7
1941	9.2	7.6	10.9	25.7	48.4	46.3	25.0	33.2	54.7	14.2	0.0	0.0
1942	7.7	8.2	8.2	35.0	223.4	250.9	94.2	39.1	33.3	23.3	14.4	10.5
1943	8.3	9.8	26.7	88.8	114.0	146.1	70.5	17.4	10.4	33.4	0.0	0.0
1944	0.1	0.0	3.6	10.7	52.8	26.7	26.2	26.1	17.5	12.7	0.1	0.0
1945	0.0	0.1	0.0	2.0	99.5	194.4	56.4	14.6	13.0	13.9	14.0	9.6
1946	8.1	6.7	10.3	34.3	108.8	128.3	47.7	16.3	20.6	21.8	19.2	14.5
1947	12.1	21.1	29.5	71.0	178.4	119.9	40.8	21.2	24.3	51.5	29.8	11.6
1948	11.0	8.7	12.7	84.8	257.6	352.8	65.8	49.0	17.5	14.6	10.6	6.9
1949	7.8	7.2	8.1	32.6	114.6	92.9	31.5	23.7	43.7	35.6	0.0	0.0
1950	0.0	0.0	2.7	43.9	133.1	196.1	89.1	29.0	12.7	19.4	22.6	22.2
1951	9.7	10.7	18.8	79.5	263.0	245.7	163.0	68.2	117.2	72.8	28.3	23.2
1952	11.7	12.6	21.2	111.8	121.7	95.6	51.3	34.2	16.9	11.8	14.1	11.7
1953	9.3	9.0	14.3	46.4	173.4	428.3	98.6	33.4	16.9	12.3	13.0	8.9
1954	8.0	10.5	10.9	35.1	190.4	178.1	111.9	37.0	34.1	32.0	20.6	17.8
1955	9.2	8.2	8.5	48.5	136.9	188.3	96.3	26.2	12.0	23.2	21.5	11.3

The low level outlet will be sized to pass the maximum flows required to meet these needs taking into account the variation in reservoir levels that might occur. The terms of reference for the preliminary engineering phase dictated that the project structures be sized to release 490,000 dam³ (400,000 ac.ft.) from the reservoir at a uniform rate of discharge over the six month period from May to October. The reservoir storage volume and surface area relationship with reservoir elevation are shown in Table 2-5 and on Plate 5.

TABLE 2-5
RESERVOIR STORAGE VOLUME AND SURFACE AREA

RESERVOIR ELEVATION	SURFACE AREA	STORAGE VOLUME
(m)	(ha)	(dam ³)
1055	20.8	225
1060	73.0	2571
1065	131.7	7,608
1070	200.4	15,764
1075	286.4	27,852
1080	401.7	44,900
1085	524.6	68,063
1090	688.2	98,381
1095	862.7	137,070
1100	1,086.4	185,750
1105	1,368.4	246,976
1110	1,689.7	323,381
1115	2,207.0	416,249
1120	2,594.6	531,784
1125	3,262.6	678,176
1128	3,687.6	782,310

Some assumptions were necessary, however, to permit the evaluation of hydroelectric power benefits that might be realized with the restraints imposed by the irrigation release patterns. Three reservoir operation scenarios were developed that bracket the range of possible release patterns daily, annually and over the life of the project. These are described in Section 14.

2.7 UPPER LIMIT OF STORAGE

Given that the average annual runoff to the site is 1,240,000 dam³ and the lowest annual runoff of 464,000 dam³ was recorded in 1944, it is reasonable to conclude that hydrologic considerations do not provide an upper limit to the volume of storage development, at least in the range of 490,000 dam³ to 750,000 dam³.

2.8 RESERVOIR FILLING

The reservoir will start to fill in the fall of 1990, after the east tunnel valve has been installed and can be operated. The west tunnel will then be closed off to allow it to be plugged and the valve to be installed. If reservoir inflows during the following year are close to the average recorded for the basin, the reservoir will fill to FSL by June 1991 with the low level releases limited to 10 m³/s during the entire period of filling. If, however, the reservoir flows are close to the driest years of record, namely, 1944 and 1977, the reservoir will take 18 to 19 months to fill; again with the reservoir outflows maintained at the proposed riparian level of 10 m³/s for the entire period.

SECTION 3

RIVER HYDRAULICS

3.1 RIVER MORPHOLOGY AND SEDIMENTATION

Impoundment of water behind a dam results in entrapment and progressive accumulation of river transported sediment within and immediately upstream of the reservoir. This process, together with a changed downstream hydrological regime, results in morphological changes to the river system both upstream and downstream of the dam. The degree and rate to which these changes will develop have been assessed for the project and have included the determination of:

- the rate of sediment deposition in the reservoir,
- the areal distribution of sediments in the reservoir,
- the river bed aggradation at the head of the reservoir, along each of the three major tributaries, and its effect of water levels, and
- the river bed degradation downstream of the dam and its effect on channel morphology, water levels and winter ice formation.

Details pertaining to the analyses covering the above determinations will be found in Appendix C.

3.2 PRESENT CONDITIONS

3.2.1 Geomorphic Features

In the area of the project site, the Oldman River has formed its valley by downcutting through glacio-lacustrine and glacial till deposits into the underlying bedrock. At the project site the river has formed a steep-walled narrow valley. The river has point bars and occasional islands.

Lateral migration rates of the river, in the vicinity of the project site, through erosion of the banks, are small compared to the upper reaches of the Oldman River as the banks contain substantial amounts of coarse grained material. In some reaches channel shifting is constrained by bedrock outcrops along a valley wall toe.

Bedrock is not exposed at the river channel surface in the vicinity of the project site. At the site, bed material consists of alluvial sand, gravel and cobbles to a depth of about 9 m. Bedrock was however encountered at a depth of 2 m below river bed level in a test pit about 1 km below the project site.

3.2.2 River Sediment

Erosional Features

Erosion of fine grained glacio-lacustrine soils provides the greater part of the sediment load in the Oldman River. These fine-grained soils lie along the lower 15 km approximately of the upper Oldman River, Crowsnest River and Castle River.

Bed Material

The bed material of the Oldman River in the reach below the project site consists of coarse gravel and cobbles. The surface material is coarser than that in the sub-surface. The average grain size distributions of these materials are shown on Plate 5.

Imbrication (i.e., alignment and overlapping) of the larger surface stones is visible indicating that the river bed is paved and highly resistant to movement. Analysis of the mobility potential of the surface stones confirm that the surface layer will remain stable except during very high floods.

Based on existing conditions the mean annual bed load of the surficial layer is 2 kt (2×10^6 kg).

Suspended Sediment

The analysis of the suspended sediment load at the project site was based on the eighteen year period from 1966 to 1983 during which data have been collected by Water Survey of Canada at the gauging station 05AA024 located near Brocket. Annual suspended sediment loads ranged widely from 7 kt in 1977 to 1108 kt in 1975. The mean value is 266 kt. Silt and clay make up about 70 percent of the suspended sediments, as shown on Plate 5.

On the average, 91 percent of the annual load occurs during May and June; these represent the two months of maximum discharge. Occasionally, most of the annual load is carried during a period of only a few days. On June 20, 1975, during

the large flood referred to in Section 2, the daily load was 53 percent of the total load for that year. It exceeded all reported annual loads except for those of 1967 and 1972.

A value of 320 kt has been adopted as the present long term mean annual total river-borne load of the Oldman River at the project site. This is 20 percent greater than the measured suspended sediment average to allow for the large variance of the data and the relatively short period of record. The annual bed load component of 2 kt is insignificant compared to the suspended load and has been ignored.

3.2.3 River Hydraulics

The Oldman River longitudinal slope through the project area averages 2.2 m/km. The discharge rating or tailwater curves at the location of the diversion tunnel outlet and at the spillway flip bucket are shown on Plate 5.

3.2.4 Ice

The natural ice regime along the Oldman River was observed during February 25, 1985, at the project site and from bridge crossings. The conditions observed are understood to be typical of those occurring each year.

The ice along the downstream reach of the river was observed to be quite smooth. There were no signs of shoving or packing, indicating that velocities at formation were probably about 0.3 m/s or less. At higher flows minor shoving or packing may occur. The warm weather at the time of the visit was causing an increase in discharge. At the end of the day water was noted on the surface of the ice at

the Monarch bridge. This was probably caused by the bridge piers and river banks preventing the ice cover from floating up with increasing river stage. A few small leads had opened at the project site along the left bank near the dam axis but there was no evidence of ice movement.

The ice regime was again observed on February 26, 1986. Chinook conditions beginning on February 23 had caused nearly the entire snowpack of about 60 cm to melt within 24 hours. In addition, unseasonable rainfall contributed to the runoff. The runoff had caused the river ice cover to break up resulting in overbank flooding at various points along the river in the vicinity of the project site. Large blocks of ice were deposited along the flood plain. The frequency of this event is not known but is believed to be rare.

3.3 PROJECT IMPACTS

3.3.1 Reservoir Characteristics

The reservoir will have three arms formed by the Oldman River, Crowsnest River and Castle River as shown on Plates 1 and 3. The reservoir storage and area curves are shown on Plate 5. At the full supply level (FSL) of elevation 1118.6 m, the reservoir has a volume of 490,000 dam³ and a surface area of 2420 ha. The capacity to inflow ratio is 0.40, based on the mean annual inflow of 1,240,000 dam³.

3.3.2 Reservoir Sedimentation

The sediment entering the reservoir has two components.

- Suspended sediment transported by the Oldman River, either as wash load or as bed material transport.
- Eroding of slopes around the reservoir rim under the action of gravity.

The adopted annual river-borne load entering the reservoir of 320 kt assumes that existing conditions of climate, land use, river hydraulics, and erosion will remain unchanged in the future after the project is in operation. It has also been assumed for the reservoir sedimentation calculations that the reservoir will be held constantly at FSL. In addition, the prediction of the sediment deposition pattern assumes that no structures will be built within the reservoir. Using these assumptions it has been calculated that, after 100 years of operation, a 6 m depth of sediment will settle against the upstream face of the dam.

Crossings of the reservoir by bridges or combinations of causeways and bridges are planned. The effect on the sediment deposition pattern of such reservoir crossings will be assessed when details of the crossings become available. Also the assessment of bank erosion around the reservoir rim by the reservoir consultant has not been included in the sedimentation estimate. The effect of these additional factors on the total sediment load entering the reservoir and sediment deposition will be documented in Appendix C, which will be published after the reservoir related works study has been completed.

Plate 5 also shows the storage volume capacity and area curves of the reservoir after 100 years of sedimentation. In terms of volume loss, the reservoir will lose annually, on the average, 0.06 percent of its original capacity. At this initial rate of loss the reservoir would have a half-life of 800 years. Three-quarters of the sediment will deposit in the Oldman River downstream of the Crowsnest River confluence. Approximate sediment deposition volumes in reaches of the reservoir based on the assumptions listed above, are summarized in Table 3-1. This table will be revised in Appendix C after the assessment has been completed.

TABLE 3-1
100-YEAR SEDIMENT DEPOSITION

RESERVOIR REACH	VOLUME DEPOSITED
	(dam ³)
<u>Above Confluence</u>	
Upper Oldman River	4,000
Crowsnest River	600
Castle River	3,400
<u>Below Confluence</u>	
Oldman River	24,000
TOTAL	32,000

3.3.3 Aggradation

Increase in bed levels of the three tributaries at the head of the reservoir due to deposition of bed load in the backwater zone will be minimal because of the low bed load transport. After 100 years the bed level in the backwater zone of the upper Oldman River will increase by about 2 m. This will cause water levels in the zone to increase by about 1 m at extreme flood flows. In the Castle River the corresponding increases will be 1.7 m and 1 m respectively. Aggradation will be insignificant in the Crowsnest River because its bed load transport is negligible.

3.3.4 Downstream Degradation

The river bed below the project site will degrade due to the entrapment of coarse sediment in the reservoir. An ultimate degradation of 0.5 m is predicted for the river bed in the reach below the project site. This was assessed using the U.S. Army Corps of Engineers HEC-6 computer model and assuming stable slopes. Assumptions made were that:

- flows will be similar to present conditions and the reservoir will be effectively run-of-river during high flow events, and
- the surficial bed material that currently protects the bed will be removed to expose the slightly smaller underlying material.

It is considered that degradation of the river bed immediately below the dam will be very small, because the bed contains a significant proportion of cobbles that only move during rare floods. It is estimated that the ultimate degradation of 0.5 m will take 25 years to occur. This would reduce water levels just below the dam by approximately 0.2 m.

Tailwater rating curves for present conditions are shown on Plate 5. The effect of the degraded bed level reduces water levels by approximately 0.2 m.

3.3.5 Ice

The natural ice regime is very stable. The ice cover forms by in situ thermal growth and not by shoving, packing or jamming. Winter flows 50 to 100 percent higher could be tolerated without appreciably affecting the ice regime. However, care will have to be taken not to suddenly increase the discharge in winter after the ice cover has formed since this could cause the river to flow over top of the ice, which in turn will result in an increase in the downstream ice cover with corresponding potential for problems at break-up. The safest course will be to maintain a constant discharge throughout the winter but a compromise may have to be considered if a peaking operation of the hydroelectric power plant is permitted.

At the site there should be no problem in handling the ice. It may be prudent to try to maintain low velocities of less than 0.3 m/s through the diversion tunnels during the winter. If this is difficult to achieve an ice boom upstream of the entrance will be constructed to prevent ice flows from moving into the tunnel.

If the rare flood event of February 1986 should occur during the diversion period the ice cover through the project site will again break up. Cofferdams will be armoured against the abrasive action of flowing ice. The design will consider that some ice jamming at the tunnel entrance should be expected but that the resulting water levels should be much less than the height of the upstream cofferdam.

SECTION 4

GEOLOGY

4.1 SITE INVESTIGATION

4.1.1 Field Program

The field program was initiated in early May 1985 and the work was essentially completed by September. The program included 37 bedrock core holes, 67 overburden holes, two bedrock exploration shafts, 105 test pits, two abutment trenches and bedrock and overburden mapping of 17 surface exposures within 500 m upstream and 800 m downstream of the dam axis. Packer tests were carried out in selected holes and piezometers have been installed to monitor groundwater conditions. Large core samples of structural discontinuities found in the rock were taken from one of the shafts and from the abutment trenches. A test tunnel was driven in the left abutment. The location of the test holes in the vicinity of the project site are shown on Plate 6. Full details of the site investigation are given in Appendices D, E, F, G, and H.

4.1.2 Rock Exploration

The bedrock core holes were drilled with Cyclone TH 60 and Failing 1500 drilling rigs, employing 3.05 m long Christensen HQ size wireline double tube core barrels and a Chrisdrill Stratopac drill bit, with air-water or water flush circulation, to retain 76 mm diameter cores. In general excellent core recovery and core quality was achieved. The length of coring in the holes ranged from 1.3 m to 123 m, for a total cored length of 2,442 m. Thirty of the holes were drilled from the uplands or valley sides, and seven from the valley floor.

Most of the holes were drilled vertically. However, three holes were inclined at 45° to investigate vertical jointing in the rock. The inclined holes were drilled adjacent to the valley sides and from the valley floor to investigate the effects of valley rebound on the near surface rock. Geophysical logging of the core holes included natural gamma, neutron, caliper, resistivity and density logs.

4.1.3 Overburden Exploration - Foundations

The overburden deposits mantling the upland areas were investigated by overburden holes; 33 drilled by a flight auger rig and six by a wet mud-rotary drill. Both disturbed and undisturbed samples were obtained from selected depths in the holes. The depth of the overburden holes ranged from 4 m to 47 m for a total length of 690 m. Some of the glacial till overburden units encountered were very dense with a significant cobble and boulder content. As a result, both drilling and sampling were difficult and in many of the augered holes, the power of the auger rig was inadequate to reach bedrock. The mud-rotary rig was brought in to overcome this difficulty.

A Becker hammer drill was used to drill 27 holes through the sand, gravel and cobble alluvium covering the valley floor. Cycloned air-return samples were augmented by split spoon and Shelby tube samples. The latter sampling methods were also used to assist in determining the alluvium-weathered bedrock contact. In six holes, bedrock beneath the alluvium was cored using a NQ size double tube core barrel in conjunction with the Becker drilling rig. Casing was set in the remaining holes for subsequent coring by the Cyclone TH 60 or Failing 1500 drill rigs. The depth of the Becker holes ranged from 4 to 29 m for a total length of 170 m plus 22 m of coring. Three of the holes were inclined at 45°.

4.1.4 Construction Materials Inventory

Twenty-eight backhoe test pits, ranging in depth from 2 m to 6.3 m, were excavated and sampled on the left and right banks at the project site. The depth capability of the track-mounted backhoe was about 6 m. Seven of the test pits were terminated in bedrock at depths of 2 m to 5.3 m. One hundred and fifty-two samples were obtained from the test pits.

Seventy-seven test pits were excavated and sampled in sand and gravel prospects upstream and downstream of the project site. These included three relatively shallow test pits in the river alluvium for the sediment and degradation analysis described in Section 3. Test pit depths were limited by caving below the water table in the alluvial deposits. Four hundred and seventy samples were recovered from the test pits in the sand and gravel deposits.

Twenty-four flight auger holes, ranging in depth from 1.2 m to 35.2 m, were drilled and sampled on the left and right banks at the project site. Plastic standpipe piezometers were installed in 14 of these auger holes. Refusal was encountered at depths as shallow as 1.2 m in some of the auger holes on the left bank terrace deposit B110, shown on Plate 15, and along the north end of the right bank. Both solid stem and hollow stem augers were used depending on test hole conditions. Four hundred and twenty samples were recovered from the flight auger holes including large bulk samples, Standard Penetration test samples and 76 mm diameter Shelby tube samples.

Twenty-five holes, ranging from 6.9 m to 29 m in depth, were drilled, using a Becker hammer drill, upstream and downstream of the project site in alluvial sand and gravel deposits. The Becker drillholes extended into the bedrock and provided information regarding the depth of the sand and gravel deposits. Seventy-seven of drill cuttings samples were recovered from the Becker drillholes.

Seven rotary holes, 24 m to 37 m in depth, were drilled on the left and right banks at the project site. These holes were logged using an electrical resistivity drillhole logger. A total of 48 samples of drill cuttings were recovered.

4.1.5 Piezometers

Twenty-eight standpipe piezometers were installed to depths of 11 m to 126 m for a total length of 1,720 m. Multiple standpipe piezometers were installed in seven drillholes to allow monitoring of groundwater pressure variations with depth.

4.1.6 Permeability Testing

Pneumatic packer permeability tests were performed in 12 core holes, for an aggregate testing length of 931 m. Typically double packer tests were performed with packer intervals of 2 m to 5 m.

4.1.7 Exploratory Shafts

Two 1.1 m diameter exploratory shafts, 26 m and 50 m deep were sunk into the bedrock beneath the valley floor using a cable tool churn rig. These were numbered SH-85-D1 and SH-85-D95 and are located in Plate 6. In the deeper shaft the exposed in situ bedrock was logged in detail and 200 mm diameter horizontal cores of bedrock containing structural discontinuities or seams were extracted from the shaft walls for testing. Entry into the second shaft has been precluded by large inflows of groundwater originating in three very pervious layers within the Basal Sandstone Sequence. A grouting and dewatering program will be undertaken in early 1986 to overcome this difficulty. The log of shaft SH-85-D95 is included in Appendix D.

4.1.8 Geological Mapping

A geological mapping program was undertaken of the bedrock exposed in the valley side, with 17 vertical exposure sections within 500 m upstream and 800 m downstream of the dam axis being mapped in detail. Survey points were established for each section to provide plan and elevation control. Oblique photographs were used as control for mapping selected structural features between the vertical exposure sections. Two trenches, one on each river bank, were excavated through thin colluvium deposits and weathered bedrock on the lower left valley slope and most of the right valley slope to allow bedrock mapping. Overburden exposures along selected reaches of the river have been inspected as part of the investigation of the buried river channel lying north of the site.

4.1.9 Geophysical Surveys

Surficial geophysical methods were employed to obtain subsurface information that could be used as an aid in the interpolation and extrapolation of drillhole 106 and other data. The main objectives of the geophysical surveys were as listed below.

- To determine the depths to bedrock, using compression wave and shear wave refraction seismic techniques. These were used on the left and right banks and in the valley bottom at the project site, and in the alluvial deposits investigated as potential sources of construction materials. Also, the depth to bedrock in the buried channel to the north of the project site was located using refraction seismic techniques.
- To determine the resistivity profiles using direct current (DC) soundings on the left and right banks to assist in locating the contact between the glacio-lacustrine deposits and underlying glacial clay till.
- To delineate granular deposits using fixed frequency electromagnetic (EM) techniques in potential borrow deposits, other than alluvial deposits. The EM survey was also carried out on the left bank to locate the thickness of the thin surficial granular deposits to the south of the dyke.
- To delineate the possible extent of the Upper Mudrock Sequence overlying the Basal Sandstone Sequence at selected locations in the valley bottom using a combination of refraction seismic and electrical techniques.

- Provision of in situ parameters for the determination of dynamic and elastic moduli.

Approximately 12 km of shear wave seismic line was surveyed: 6 km at the project site and 6 km in potential deposits. At the dam site, 58 DC soundings were carried out for a coverage of approximately 5.8 km.

The results are presented in Appendix G as subsurface geophysical sections along the lines surveyed, showing the depth to bedrock and intermediate layers. Contour maps of the elevation of top of bedrock were prepared for the left and right banks, based on the geophysical sections.

4.1.10 Test Tunnel

A test tunnel was driven into the left bank downstream of the toe of the dam. The objectives of the test tunnel were:

- to demonstrate the feasibility and economics of excavation of the interbedded rocks at the site,
- to evaluate different methods of primary tunnel support,
- to ascertain the time dependent properties of the mudrocks, by measurement of their deformation with time,
- to collect geotechnical data for use in design of the tunnels and other project components, and
- to collect hydrogeological data to assess the drainage characteristics of the dam foundation.

The test tunnel was at the same elevation and parallel to the diversion tunnels and intersected rock types and structures which should be representative of the ground through which the diversion tunnels will be driven. The test tunnel consisted of a 60 m access section, excavated to 3.5 m diameter, followed by a 10 m transition section to a 30 m long test chamber excavated to 7.5 m diameter simulating the full diversion tunnel size. The excavation of the test tunnel simulated the driving conditions in the diversion tunnels and methods of achieving a stable drill-and-blast excavation with minimum overbreak were attempted. The use of a tunnel boring machine (TBM) was not feasible to drive the test tunnel as the high cost of mobilization could not be justified for such a short length of tunnel.

A variety of tunnel support systems were evaluated in the test chamber. These included resin grouted rock bolts, split-set rock bolts, Swellex rock bolts, rock dowels, unreinforced shotcrete, fibre reinforced shotcrete, and silica fume shotcrete.

Instrumentation was installed in the test chamber to monitor the rate of deformation of the tunnel walls and to record piezometric pressures in the ground surrounding the test tunnel, both during and after excavation. The instrumentation consisted of drillhole extensometers, convergence points, and standpipe piezometers. In situ tests in the tunnel included rock bolt pull-out tests, shotcrete strength and adherence, and deformation measurements.

Rock exposed in the walls of the tunnel was mapped. Core samples were be taken of the shotcrete and of the rock encountered.

Records of the test tunnel work will be made available to diversion tunnel tenderers. Prospective contractors were given the opportunity to inspect the test tunnel while it was being driven. Detailed results of the test tunnel program are contained in Appendix E.

4.1.11 Laboratory Testing

At the outset of the field program a temporary laboratory was set up in Pincher Creek to undertake index laboratory tests as the drilling program progressed. Once the majority of the field drilling had been completed, the field laboratory was closed and laboratory testing continued at permanent laboratories. The laboratory testing completed to November 1985 is summarized in Table 4-1. Some further testing is in progress.

The tests are being conducted in accordance with standard procedures established by testing and technical organizations such as ASTM, AASHTO, ISRM, and PFRA, or in accordance with recommendations given in technical literature.

Much of the testing program effort was devoted to direct shear testing of structural discontinuities and seams in the claystones in the dam foundation which will control dam and spillway stability. Due to irregular and non-planar shear surfaces that often develop during testing, considerable ramping occurred during the residual strength determination phase of the tests. Ramping is cyclic vertical movement of the upper portion of the test specimen, due to the relative displacement of the two halves of the specimen along a

TABLE 4-1
LABORATORY TESTING

TEST	NUMBER
Natural Water Content	1075
Visual Classification	663
Atterberg Limits - non-blenderized	81
- blenderized	158
Sieve Analysis	135
Hydrometer Analysis - non-blenderized	77
- blenderized	51
In situ Density	8
X-ray Diffraction	29
Slaking	31
Swelling Pressure	3
Swelling Strain	3
Water Soluble Sulphates	18
Consolidation	5
Unconfined Uniaxial Compression	108
Triaxial: Consolidated undrained with pore water pressure measurement	27
consolidated drained	3
Direct Shear (Peak and Residual)	67
Dynamic Modulus and Poisson's Ratio	9
Point Load Tests	39
Cation Exchange Capacity	5
Organic Content	4
Dispersion	2
Permeability	2

failure surface that is slightly inclined and not exactly parallel to the shear box. Asperity correction procedures have been used to account for these effects. These are discussed in Appendix D. Due to the importance of the direct shear testing, large 100 mm shear boxes were used in addition to standard 50 mm boxes.

4.2 REGIONAL GEOLOGY

A detailed description of the regional geology is given in Appendix D. A summary is presented in this section.

The project site is situated in the Alberta Plain physiographic region near its boundary with the Rocky Mountain Foothills. Subsurface conditions include glacial and postglacial soils overlying bedrock. Major rivers and streams have eroded through soil and into bedrock. A regional geological map is presented on Plate 7.

Glacial sediments overlie bedrock and include sequences of clay till and glacio-lacustrine silt and clay. Postglacial sediments include alluvial gravel and sand in terraces and floodplains along the river and stream valleys, colluvium in the valley slopes, and a discontinuous blanket of eolian silt in the upland areas.

Two bedrock formations have been mapped in the area, the Porcupine Hills Formation and the underlying Willow Creek Formation. Regionally, the Willow Creek Formation consists of poorly cemented sandstones, bentonitic mudrock, and occasional bentonite seams. The Porcupine Hills Formation consists of sandstones and mudrocks. The sandstones of this

formation are more prevalent and more cemented than in the Willow Creek Formation. At the site the mudrocks of both formations are not bentonitic and no bentonite seams are known. The contact between the two formations has been tentatively established, by palynological examination of core samples, (the use of pollen and spore microfossils to determine stratigraphic succession) at a depth of approximately 50 m below the river valley floor (Jerzykiewicz and Sweet (1985)). The Porcupine Hills Formation underlies the Quaternary surficial deposits.

The Alberta Syncline is the only major bedrock structure in the area. It is asymmetric with a shallow dipping eastern limb and a steep dipping western limb where it approaches thrust faults of the Disturbed Belt. The axis of the syncline trends southeast and appears to lie several kilometres to the east of the project site. Mesoscopic fabric elements in the rock mass include bedding, joints and shears. The regional dip is gently northeast but in the vicinity of the project site, the dip is negligible, reflecting its location near the axis of the Alberta Syncline.

4.3 SITE SURFICIAL GEOLOGY

4.3.1 Stratigraphy

Surficial materials present at the site include glacio-lacustrine deposits, glacial drift, alluvium and colluvium. These are described in detail in Sections 4.9 and 4.12, under the headings of Dam Foundation and Construction Materials respectively.

The glacial drift includes two clay till sheets, and is overlain by a blanket of glacio-lacustrine silt and clay. The drift occupies the upland areas and fills a buried valley north of the project site.

Alluvium consists of gravel and sand often containing cobbles and boulders. These materials occupy terraces along the Oldman River valley slopes and the floodplain of the river.

Colluvium is derived from weathered bedrock and glacial drift. The material has a till-like appearance but contains a high proportion of angular bedrock clasts and occasionally blocks of bedrock. It is generally thin and overlies weathered bedrock.

4.3.2 Buried Valleys

Buried valleys have been located north and south of the dam site. The one lying to the south of the project site is being investigated in conjunction with reservoir studies. An investigation of the buried valley lying north of the site is reported in detail in Appendix H.

The bedrock topography, as interpreted from aerial photographs, limited drilling and geophysical data, suggests the thalweg of the valley trends roughly east-southeast. The valley intersects the present river bank upstream of the project site and along the north bank of the Oldman River valley between 5 and 8 km downstream of the dam. The floor of this valley appears to be at an elevation of 1,058 m based on drilling data. The bottom width of the valley is approximately 300 m and the top width is approximately 5,500 m. Both slopes appear to be approximately 25 horizontal to 1 vertical.

Water-well and drilling data from previous studies indicate bedrock at the bottom of the buried valley is overlain by 1 to 2 m of granular soils which are in turn overlain by more than 50 m of undifferentiated glacial drift. The bottom elevation of these granular soils is 1,058 m. Granular soils also overlie the bedrock contact along the left bank of the Oldman River between 1.5 and 2.5 km above the project site and again approximately 5.5 km upstream of the site. The granular soils consist of a range of material from sandy gravel to silty sand. These are considered to be glacial outwash rather than the Saskatchewan Sands and Gravels which are of preglacial age. The appearance of the basal granular materials, together with the relatively narrow crest width of the buried valley, suggest that the valley is interglacial, rather than preglacial.

An extensive trenching program formed the major thrust of the investigation. A few stratified sand and gravel pockets were identified within the thick undifferentiated glacial drift sequence overlying the basal granular materials. None of significance was found in outcrop or in exploratory trenches. On this basis, no contiguous beds of stratified materials are considered to be present within the thick sequence of glacial drift filling the buried valley and leakage of water from the reservoir through the overburden is expected to be negligible. An appraisal of the rock mass and spacing of joints and openings in outcrop indicates a similar conclusion for the rock underlying the buried valley.

4.4 SITE BEDROCK GEOLOGY

4.4.1 Bedrock Stratigraphy

Rocks underlying the abutments and foundation of the dam are part of the Porcupine Hills and Willow Creek Formations. They consist of interbedded sequences of fresh water sandstones and mudrocks. It is believed they were deposited in a meandering fluvial system where sandy deposition was restricted to main channels or to partially or completely abandoned meander loops. Deposition of fines occurred on levees and in flood basins.

The distribution of the major stratigraphic units comprising the Porcupine Hills Formation and Willow Creek Formation as encountered during exploration at the project site are shown in Table 4-2.

TABLE 4-2
PROJECT SITE STRATIGRAPHY

FORMATION	MAJOR STRATIGRAPHIC UNITS
Porcupine Hills	Upper Mudrock Sequence
	Basal Sandstone Sequence
-----DISCONFORMABLE CONTACT-----	
Willow Creek	Lower Mudrock Sequence

The Lower Mudrock Sequence consists of interbedded siltstone, claystone and sandstone. It is the lowest of the three major units. It underlies the Basal Sandstone Sequence throughout the site area with the elevation of the contact at approximately 1005 m in the foundation of the dam and rising in a downstream direction.

The Basal Sandstone Sequence is predominantly sandstone with some mudrock. Two important minor units, Structural Unit Nos. 1 and 2, have been identified within the Basal Sandstone Sequence and consist mostly of siltstone and claystone. The overall sequence has a relatively uniform thickness of 38 m in the dam foundation, with the lower and upper contacts at elevations 1005 m and 1043 m, respectively. Structural Unit No. 1 is centered at about elevation 1030 m in the dam foundation and is continuous throughout the site. South of the dam axis and under the left abutment it ranges in thickness from 5 to 7 m in the river valley. Upstream of the dam and under the right abutment it has an overall range in thickness from 1 to 10 m. Structural Unit No. 2 is discontinuous. It underlies the left half of the dam foundation and the left abutment. It is centered at about elevation 1020 m and ranges in thickness from 2 to 3 m.

The Upper Mudrock Sequence consists of interbedded siltstone, sandstone and claystone. The lower contact with the Basal Sandstone Sequence is erosional. It lies at an elevation of about 1043 m in the dam foundation but ranges over the site from elevations 1036 m to 1046 m. The Upper Mudrock Sequence extends upwards to the bedrock surface, where it is overlain by Quarternary surficial deposits. The elevation of the highest bedrock surface is about elevation 1125 m. The Upper Mudrock Sequence also includes two minor units, both of which

are channel facies; Sandstone Channel Nos. 1 and 2. Sandstone Channel No. 1 is an eastward thinning wedge that outcrops high on the left abutment and extends at least 150 m eastward beneath the uplands. It is centered at about elevation 1090 m and has a maximum thickness of 16 m a short distance east of the outcrop position. Sandstone Channel No. 2 is situated at or near the basal contact of the Upper Mudrock Sequence. The present distribution is locally affected by erosion along the Oldman River valley, however it is thickest near the bottom of the right abutment where it is up to 18 m thick, and thins beneath the dam foundation to the left abutment, where it is 2 m to 5 m thick. Although drilling information indicates this unit is absent well upstream of the dam, an equivalent sandstone channel sequence is believed to be present between the upstream diversion tunnel portals and the Gifford tributary valley where thick sandstone outcrops are present near river level.

Plates 11 to 14 inclusive show generalized geological sections through the dam axis, river valley, tunnels, and spillway, respectively.

4.4.2 Rock Types

There are three main rock types at the site. These are claystone, siltstone and sandstone. The sandstone may be subdivided into two types, an overbank and shallow channel facies, and a deep channel facies. The diagnostic features of each rock type are described in the following summaries and the results of classification test data on samples taken from them are presented on Plate 8.

Claystones

- grey to dark grey, occasionally black or green
- calcareous
- very silty (silt sizes probably comprise a higher percentage than clay sizes in most claystones)
- occasionally carbonaceous
- calcrete nodules are locally common
- medium plastic clay (CI), occasionally ranging into low and high plastic clay (CL and CH).
- x-ray diffraction analysis of the clay size fraction shows:
 - quartz as the predominant mineral
 - calcite, dolomite, kaolinite and mica are secondary minerals
 - smectite and chlorite are present as trace amounts in most samples.
- generally shows a "slow" rate of slaking, although some samples are rated as "very fast"
- develops a fissured structure as a result of weathering
- strength descriptions vary from very weak to strong.

Siltstones

- light grey to dark grey, occasionally green or black
- calcareous
- ranges from clay rich to sand rich
- occasionally carbonaceous
- calcrete nodules are locally common
- medium plastic clay (CI), commonly ranging from non plastic, to low plastic silt or clay (ML or CL).
- x-ray diffraction analysis of the clay size fraction shows:
 - quartz is the predominant mineral
 - calcite, dolomite, kaolinite and mica and secondary minerals
 - smectite and chlorite are present as trace amounts in the majority of samples.
- the very significant degree of overlap in plasticity and mineralogy between claystones and finer grained siltstones underscores the difficulty in discerning between the two rocks.
- occasionally develops a fissured structure as a result of weathering
- sandier units often show ripple lamination structures
- strength descriptions vary from very weak to very strong

Sandstone (overbank and shallow channel facies type)

- light grey to dark grey, some pale brown and pale greenish grey
- generally fine grained (mostly 0.125 mm to 0.25 mm in size), with occasional thin medium to coarse grained zones
- often silt rich
- poorly graded (SP) occasionally silty (SM)
- calcrete nodules occasionally present
- thin rip-up clast conglomerate composed of mudchips and/or carbonaceous debris occasionally present
- the clastic components are:
 - quartz (30%), sub-angular to sub-rounded grains
 - chert (30%), sub-rounded grains
 - carbonate (30%), sub-angular to angular grains
 - feldspar, chlorite, white mica, biotite, apatite, tourmaline, opaques, organic matter, zircon and rock fragments (10%).
- the cement components are:
 - carbonate (most abundant)
 - kaolinite
 - silica
- very low porosity (1% to 5%)
- laminated and bedded structures common
- strength descriptions vary from moderately strong to very strong

Sandstone (deep channel facies type)

- light grey to grey, some pale brown and pale greenish grey
- fine, medium and coarse grained
- poor to well graded (SP) to (SW)
- beds of rip-up clast conglomerate composed of mud-chips and/or carbonaceous debris common
- occasionally conglomeratic
- occasional detrital coal pockets
- clay balls and claystone or siltstone fragments common
- the clastic components are:
 - quartz (30%), sub-angular to sub-rounded grains
 - chert (30%), sub-rounded grains
 - carbonate (30%), sub-angular to angular grains
 - feldspar, chlorite, white mica, biotite, apatite, tourmaline, opaques, organic matter, zircon and rock fragments (10%)
- the cement components are:
 - carbonate (most abundant)
 - kaolinite
 - silica
- very low porosity
- bedded structures common, occasionally laminated structures

- strength descriptions vary from strong to very strong
- unconfined compressive strengths vary from 40 to 100 MPa.

4.4.3 Detailed Stratigraphy

For the purpose of dam stability design studies, the main stratigraphic units identified during the geological investigation of the project site were subdivided into smaller rock units on the basis of rock type. This was done by study of drillhole data. These rock units were derived from several upstream-downstream oriented geological sections. The units are not shown on Plates 11 to 14 inclusive. The rock units are listed in Table 4-3 to illustrate the variability of the rock types that comprise the bedrock at the project site. Units J1 to J11 inclusive comprise the Upper Mudrock Sequence. The J units were primarily subdivided on the basis of the several bands of calcrete nodules found within the sequence. Units K1, K2, L and M comprise the Basal Sandstone sequence. The Lower Mudrock Sequence underlying the Basal Sandstone sequence was identified as Unit N; this unit was not studied in detail, as it lies below the depth that will affect dam stability.

The continuity of the individual sandstone, siltstone and claystone beds making up the rock units units J1 to J11 inclusive, K1, K2, L and N generally varies across the site. The beds comprising units K1 and M, being almost 100 percent sandstone of uniform characteristics, are found in all drillholes and hence are assumed to be relatively continuous.

TABLE 4-3
SUMMARY DESCRIPTIONS OF ROCK UNITS AND
CORRELATION TO STRATIGRAPHIC UNITS

STRATIGRAPHIC UNIT	ROCK UNIT	DESCRIPTION	MODULAR	NON- MODULAR
Upper Mudrock Sequence	J1	Mudrocks (80%) with interbeds of sandstone (20%)	x	
	J2	Mudrocks (60-95%) with interbeds of sandstone (5-40%)		x
	J3	Mudrocks (80-95%) with interbeds of sandstone (5-20%)		x
	J4	Mudrocks (0-100%) varying with sandstone (0-100%)	x	
	J5	Mudrocks (0-100%) varying with sandstone (0-100%)		x
	J6	Mudrocks (60-100%) with interbeds of sandstone (0-40%)	x	
	J7	Mudrocks (60-95%) with interbeds of sandstone (5-40%)		x
	J8	Mudrocks (60-100%) with interbeds of sandstone (0-40%)	x	
	J9	Mudrocks (0-100%) varying with sandstone (0-100%)		x
	J10	Mudrocks (0-100%) varying with sandstone (0-100%)	x	
	J11	Mudrocks (0-100%) varying with sandstone (0-100%)		x
Basal Sandstone Sequence	K1	Sandstone (100%)		x
	K2	Sandstone (60-100%) with occasional interbeds of mudstone (0-40%)	x	
	L	Sandstone (10-90%) varying with mudstone (10-90%)	x	
	M	Sandstone (100%)	x	
Lower Mudrock Sequence	N	Mudrocks		

4.5 BEDROCK STRUCTURE

The project site is situated west of the axis of the Alberta Syncline. No other macroscopic structures are known in the vicinity. Mesoscopic structures include stratiform structures, joints, stress relief fractures and shear features.

4.5.1 Stratiform Structures

Bedding and lamination structures are well defined. Although there is significant variation in dips on a local scale because of the influence of current bedding structures, the overall dip of the bedrock at the project site is small and appears to be between 0.5° and 1°, in the upstream direction. Sandstone units typically have the thickest beds, ranging from the order of a few millimetres to massive beds more than 10 m thick. Many sandstones show cross-stratification. Bedding structures in mudrocks generally range from a few millimetres to more than 1 m thick.

4.5.2 Joints Characteristics from Surface Mapping

Horizontal bedding joints pervade all rock types. Vertical joints are most prevalent in sandstone and sandy siltstone, while joints dipping 50° to 60° occur almost exclusively in claystone. There are four significant joint sets. In order of predominance they are oriented at a dip and in a dip direction of:

02°/030°

88°/207°

89°/129°

55°/207°

The dam axis is oriented at 108° . Therefore, the vertical joint sets strike at angles of 9° and 69° to the dam axis. Hairline fractures and horizontal joints occur in exploratory shaft SH-85-D95 below elevation 1052 m.

The vertical joints are very distinct in the sandstone exposures in the upper abutments. The joints are continuous over distances of upwards of 5 to 10 m and are usually open at the ground surface. Joint surfaces are usually clean, rough and dry.

Considerable opening of joints in the sandstone has occurred in the abutments of the dam and is suspected in the foundation of the structure as well.

4.5.3 Joint Characteristics from Drillhole Data

The joint frequency data derived from the drillhole logs were analysed by considering average joint frequencies in numbers of joints per metre (jt/m) for 10 m intervals from elevation 1,120 m to 960 m.

The analysis suggests the entire foundation can be zoned horizontally. The joint frequency varies from, in excess of 2 jt/m, close to bedrock surface, to between 0.1 and 2 jt/m below. The frequency depends on depth and rock type.

The calculated average joint frequencies are considered low estimates of the true in situ joint frequency, considering that most of the drillholes and the major joint sets are vertical. A more accurate estimate is suggested from the joint frequency in the inclined drillholes where the values

are 1.5 to 3 times the average joint frequency. It appears from the drillhole data that the valley bottom as a whole is only slightly more jointed or fractured than the abutments. It is known from shaft SH-85-D1, which is located in the centre of the valley, that there are pervious zones in the rock. These are probably related to valley rebound deformations and will therefore be a localized phenomena.

4.5.4 Seams and Zones of Slickensides

Numerous thin, subhorizontal seams of potentially low shear strength were observed in the rock cores and in the exploratory shaft. They are generally concordant with the bedding, and are usually found to be associated with individual mudstone beds. The seams in general are most numerous in the mudstone-rich units J1 through J11 and unit L. They are seldom encountered in the pure sandstones. The seams contain silty clay, clayey silt or comprise fragmented or brecciated rock. The thickness of the seams rarely exceeds 7.5 cm. In some cases the seams do not contain silty clay or clayey silt material, but consist of softened mudstone. Detailed characteristics of the subhorizontal seams are given in Section 4.8.

Zones of slickensides were also observed in drill core. These are zones where numerous multidirectional, generally steeply inclined, wavy slickensided surfaces exist. The core found to contain these surfaces generally remained intact and hence the surfaces are considered discontinuous. The thickness of the zones of slickensided surfaces varies but they are generally less than 1 m.

The origin of these seams and zones of slickensides in the bedrock at the project site could be related to bedrock load reduction and stress relief which occurs during river downcutting and valley formation. This results in uplifting of the valley floor and inward movement of the valley walls, which in turn cause relative movement or shearing along bedding planes. However the presence of the massive Basal Sandstone Sequence would tend to inhibit, or at least limit, vertical strain. Other mechanisms such as tectonic deformation, compaction, creep, frost wedging, glacial overriding and glaciotectonic deformation, may also explain the structural discontinuities.

4.6 BEDROCK SURFACE TOPOGRAPHY

The bedrock surface is at between elevation 1,095 m to 1,100 m and 1,110 m to 1,120 m in the left and right river banks, respectively. The approximate overburden thicknesses in the left and right river banks are therefore 25 m and 10 m respectively. In the valley bottom, the overburden generally varies from very thin at the base of the abutment slopes to a maximum of 15 m beneath the terrace on the right side of the valley. The lowest bedrock surface in the valley bottom is at approximately elevation 1,044 m.

The results of the drilling and the geophysical studies, have enabled the bedrock surface topography in the valley bottom, and on the abutments to be determined. In the valley, a shallow bowl-shaped bedrock surface exists with a narrow thalweg between elevations 1,044 m and 1,045 m. The thalweg is in the middle of the valley in the upstream half of the

dam foundation and is in the right half in the downstream half of the foundation. Along the dam axis, the bedrock surface slopes gently beneath the river and more steeply on the right side.

The overburden in the river valley, which comprises alluvial sand and gravel and colluvial clay, silt and sand is estimated to be about 9 m deep in the right bank and 3 m to 4 m deep in the left bank.

4.7 BEDROCK PERMEABILITY

Water pressure testing was performed in twelve of the drillholes. Tests were done in stages of increasing pressure up to a pressure equivalent of 4 kPa/m of overburden, in intervals of 3 m to 5 m. For use in grout curtain design, the test results are best related to a base value of 1×10^{-5} cm/s or more preferably to one Lugeon l/min/m at 1 MPa which is the limit of practical groutability. A generalized picture of the rock permeability emerged from an analysis of the test data.

Permeability of the rock above elevation 1070 m in the river banks is generally high, ranging up to 8×10^{-3} cm/s. Between elevations 1070 m and 1040 m the permeability values are significantly less than those above elevation 1070 m. The values range from less than 1×10^{-5} cm/s to 4×10^{-4} cm/s. The left bank is apparently more pervious than the right bank.

Permeability values in the Basal Sandstone Sequence in the the abutments range from less than 1×10^{-5} cm/s to 2×10^{-4} cm/s. Permeabilities below elevation 1040 m in the

valley bottom, however, are generally higher, ranging up to 6×10^{-3} cm/s. This increase in permeability, although not reflected in the joint frequency observed in the drill core, is consistent with the valley rebound phenomena. Below elevation 1010 m permeabilities vary from less than 1×10^{-5} cm/s to 5×10^{-4} cm/s.

Existing groundwater levels in the abutments are generally low, being in the range of elevation 1,050 m to 1,060 m.

4.8 BEDROCK FOUNDATION SHEAR STRENGTH

4.8.1 Seams

A detailed assessment was made of the strength of the bedrock that will make up the foundation of the dam and spillway. It was known, from other site investigations for large dams in similar rock formations in the Alberta Plain physiographic region, that subhorizontal seams or discontinuities of potentially low shear strength can occur within the rock mass and that these can be continuous over distances large enough to have a significant influence on the stability of water retaining structures founded upon them. The foundation investigation of the project site was directed towards obtaining data necessary to determine if such seams exist in the foundation of the dam and spillway, to map their continuity, and to measure the shear strength that is available along them.

During drill core logging, discontinuities in the rock core were described in detail. The depth, angle to core axis, fillings, and surface characteristics were recorded. Specific discontinuities believed to represent zones of the

foundation bedrock having a lower shear strength than the rock mass, were identified from these data using the criteria listed in Table 4-4. Discontinuities of type 1 or 2 containing silt or clay were considered to represent the rock in the foundation with potentially the lowest shear strength. Their only distinction is orientation. The orientation of type 1 is known to be subhorizontal. The orientation of type 2 is unknown due possibly to the thickness of the seam, or to it not being reported. Types 3 and 4 are zones of fragmented rock which may also indicate in situ zones of lower shear strength. The same orientation distinction was applied for types 3 and 4. Finally, type 5 represents core loss zones which may also indicate in situ discontinuities. The discontinuities were further classified with respect to the thickness of the zones; those greater than 75 mm thick, those between 75 mm and 5 mm thick and those less than 5 mm thick.

TABLE 4-4
DESCRIPTION OF SEAMS

TYPE NO.	INFILLING	ORIENTATION*
1	Silt or clay with or without rock fragments	Subhorizontal
2	As above	Unknown
3	Rock fragments without silt or clay	Subhorizontal
4	As above	Unknown
5	- - - - - Core Loss Zones - - - - -	

* Orientation in situ. Occasional seams that dip $>20^\circ$, were excluded.

The drill core discontinuities were plotted to scale on three geological sections taken approximately parallel to the river, one through the river valley and one through each abutment. Based on these plots, the seams were inferred throughout the foundation rock mass, assuming the seams to be concordant with the bedding. Practically all such postulated seams are wholly within mudstone beds, and appear to occur along geological contacts as well as within rock units. The frequency does not appear to change with depth.

In the abutment sections, the seams occur in nearly all rock units and appear to be more frequent and more continuous on the left abutment than on the right abutment. In the valley bottom the seams are concentrated in units J10, and J11 and L.

The plotted data suggest that several seams are continuous in the north-south direction over distances as great as 700 m in the left abutment. On the right abutment, most seams are continuous over only 200 to 300 m lengths, although two approach 800 m long. Similarly in the valley bottom, continuities of up to 800 m long were inferred along several seams.

The appearance of the seams in the core suggests that they have been caused by relative movement, or shear deformation. The soft fillings have therefore been generated in situ and are not the result of deposition in open voids.

The reason for the apparent differences in the seam characteristics from one abutment to another is not known. Greater stress relief and movement due to lower bedrock in the left abutment may explain the difference.

The seams observed in the walls of shaft SH-85-D95 are well defined, concordant with bedding, continuous, and contain clay or silty clay of varying plasticity and consistency. Shaft SH-85-D95 is located at the bottom of the right abutment close to the dam axis as shown on Plate 6. The seams are fairly consistent in thickness around the shaft walls. Thicknesses of the different seams vary from less than 1 mm to 12 mm. The seams are often bounded by subsidiary continuous or discontinuous fractures subparallel to the main seam. The total thickness of the fracture zone plus central seam can approach 4 cm to 6 cm. Many of the seams are planar. Others are slightly undulating within the sample length. No slickensides are apparent on the seam surfaces.

4.8.2 Geometry of the Seams

The geometry or planarity of the seams postulated in the foundation rock was studied to estimate a roughness component of the shear strength available along the seams. The shear strength of the seams is discussed in Section 8. In most cases from the drillhole data, the inclination of a variety of possible slip surfaces through a postulated seam did not significantly differ from the bedding dip. This suggested a low value of roughness, which appears to be logical for shearing along bedding planes of a thinly bedded, fine grained, rock sequence with a lower shear strength than the surrounding rock mass.

Another assessment of roughness was made by estimating the amount of undulations observed on individual seams in the shaft. Average roughness is estimated as 0° for unit L and 1.5° for units J10 and J11 in the dam foundation. Similarly, the average roughness is estimated as 2.5° for units J7 and J8 in the lower abutment area.

4.9

DAM FOUNDATION

The subsurface conditions in the river valley were investigated for suitability as a dam foundation. An envelope of the river valley alluvium is shown on Plate 8. Classification data on the overburden on the uplands is presented on Plates 9 and 10.

The valley bottom alluvium mainly consists of well to poorly graded sandy gravel with traces of silt and clay. Laboratory test results give between 0 to 9 percent of the particles passing the No. 200 sieve, D_{10} ranging between 0.1 to 6 mm and gravel size particles ranging between 58 to 92 percent. Standard Penetration test "N" values vary between 15 and 90. Based on the gradation of the samples the permeability of this deposit is estimated to be very high. The alluvium is overlain by approximately 1 m of silty sand. The depth of the river alluvium varies from 2 m to 15 m.

Along the right bank upstream of the dam centreline there is a significant deposit of colluvium. The colluvium consists of clayey silty sand and clayey silt. The Standard Penetration test "N" values in the colluvium vary between 4 and 15. An angle of shearing resistance of 25° has been assigned to the colluvium in situ.

4.10

TUNNEL CONDITIONS

The Upper Mudrock Sequence of the left bank along the tunnel alignment includes the two major sandstone channels separated by interbedded siltstones and claystones with some minor sandstones. The twin diversion tunnels will be excavated under a maximum cover thickness of about 67 m.

Plate 13 shows the geology close to the alignment selected for the preliminary design. It can be seen from this section that the tunnel inlet invert at about elevation 1,048 m is above the thick Basal Sandstone Sequence, but will encounter Channel Sandstone No. 2 close to the outlet portal. Due to the stratigraphic variation which exists in the Porcupine Hills Formation, it is anticipated that other channel sandstones of comparable thickness could be intersected by the tunnel. Such sandstones might not have been encountered during the current investigations due to the spacing of the drillholes. Elsewhere the tunnels will be excavated within the Upper Mudrock Sequence. The ground above the tunnels into which the rock reinforcement will be installed, comprise an interbedded sequence of claystones, siltstones and thin sandstones with brecciated or sheared zones parallel to the horizontal bedding. Discontinuities are widely spaced but continuous in the sandstones and more closely spaced but likely less continuous in the mudrocks. Rock strength ranges up to 132 MPa in the sandstones, between 26 MPa and 86 MPa in the siltstones and between 5 MPa and 46 MPa in the silty claystones. Claystones at some horizons exhibit extreme slaking and have uniaxial strengths less than 10 MPa.

Permeability values and records of the existing phreatic surface at the tunnel horizon are sparse but it is concluded that inflows to the tunnel excavation through major joints could be possible. The tunnels will be driven entirely below the present phreatic surface in saturated ground.

Inlet and outlet tunnel portals will be developed within the Upper Mudrock Sequence and overlying Sandstone Channel No. 2 respectively. The sandstones stand near vertically and the mudrocks support a modest veneer of up to 4 m of colluvium. Stress relief joints are developed both parallel to the

valley and obliquely to the slopes; spacing is close to very close. The test tunnel will show the distance into the slope that the stress relief joint pattern persists. There is no existing instability at the portals.

4.11 SPILLWAY FOUNDATION

The spillway headworks and chute will be founded on mudrocks of the Upper Mudrock Sequence as shown on Plate 14. The flip bucket will be partly in the Sandstone Channel No. 2 and will be founded in the Basal Sandstone Sequence.

4.12 CONSTRUCTION MATERIALS

4.12.1 General Assessment

The surficial glacio-lacustrine deposits occur on both left and right banks and consist mainly of stratified and laminated, medium to highly plastic, silty clay and clay. The thickness of this unit encountered in drillholes and test pits ranges from 7.8 to 9.4 m on the left bank and from less than 1 m up to 8.3 m on the right bank. In the southeast part of the left bank, the glacio-lacustrine deposit also includes silty fine sand strata.

The glacio-lacustrine deposit is underlain by clay till and silty clay till of medium to high plasticity. Two separate till sheets have been identified on the left bank where the total glacial drift thickness increases toward the buried valley to the north of the project site. The total till thickness on the left bank ranges from 19 m to more than 35 m. Glacial till also underlies the glacio-lacustrine silt and clay on the right bank. However, the bedrock surface

occurs at a higher elevation under the right bank and therefore the total till thickness is considerably less than on the left bank. Till thickness decreases toward the north end of the right bank.

Numerous gravel deposits occur within, and adjacent to, the proposed reservoir and also downstream of the project site as shown on Plate 15. The gravel deposits within the investigation area include alluvial deposits along the valley bottom of the Oldman River, Castle River and Crowsnest River, river terrace deposits along the Oldman River valley and glacial outwash or deltaic deposits south of the valley between the project site and the Summerview Bridge. In general, the alluvial gravels are the most coarse-textured and contain a significant amount of cobbles and boulders. The terrace deposits also contain a significant amount of cobbles but less boulders than the alluvial deposit. The outwash or deltaic deposits downstream of the site are finer textured and contain a relatively high amount of sand with less gravel and cobbles.

Riprap quarry prospects have been identified immediately west of the project site at the Buffalo Jump and in several localities in or near the Rocky Mountains as shown on Plate 16. The Buffalo Jump area consists of sandstone. The best prospects near the Rocky Mountains are composed of limestone.

The estimated volume of material in each deposit, that will be suitable for use as a construction material is listed in the table on Plate 15. Details of the investigation and laboratory testing are contained in Appendix F.

4.12.2 Impervious Material

The two areas which appear to be most suitable for obtaining impervious material are referred to as the "left bank borrow" and "right bank borrow" areas on Plate 15. The left bank borrow area is north of the dyke and east of the tunnel intake portal. This borrow area is located in the reservoir area and can be extended northward if required. The right bank borrow area is situated southwest of the spillway.

The surficial glacio-lacustrine silt and clay must be excavated from either borrow area before the underlying glacial till can be removed.

Glacio-Lacustrine Deposits

The depth of the glacio-lacustrine deposits varies from 1 to 15 m on the left uplands and from 2 to 6 m on the right uplands. The glacio-lacustrine deposits are extremely variable. However some will be suitable for use in the dyke and in the random fill zones in the dam, with a minimum of moisture conditioning.

Tests were performed on compacted samples to assess their suitability as damfill materials. The test specimens were obtained by mixing individual samples of glacio-lacustrine deposits from test holes and test pits. A summary of the test results is listed in Table 4-5. Some test data are plotted on Plates 9 and 10.

Glacial Till

The glacio-lacustrine deposits are underlain by glacial clay till. This glacial till contains layers of clayey gravels and sandstone cobbles and boulders.

TABLE 4-5
GLACIO-LACUSTRINE DEPOSITS - TEST RESULTS SUMMARY

TEST	LEFT BANK	RIGHT BANK
In situ moisture content		
Average	17.0	19.4
Range	6.5 - 32.2	5.1 - 32.2
Atterberg Limits		
Liquid Limit	50	77
Plastic Limit	23	26
Plasticity Index	27	51
Grain Size Distribution (%)		
Clay	46	67
Silt	45	30
Sand	6	3
Gravel	3	0
Standard Proctor Compaction		
Max. Dry Density (kg/m ³)	1,665	1505
Optimum Moisture Content	21	25.5
Compression Index		
Virgin curve	0.24	0.3
Rebound curve	0.08	0.13
Effective cohesion (kPa)	61	10.5
Effective Angle of Shearing		
Resistance	19°	17.5°
Undrained Shear Strength (kPa)	153	162
Specific Gravity	2.74	2.79
Dispersion	ND1	ND1
In situ density (kg/m ³)	2,060	1,974
Permeability (cm/s)	1 x 10 ⁻⁷	3 x 10 ⁻⁸

Tests were performed on compacted samples to evaluate their properties as potential damfill. These were obtained by mixing individual samples of clay till deposits from test holes and test pits. A summary of the test results are listed in Table 4-6. Some test data are plotted on Plates 9 and 10.

4.12.3 Granular Materials

Sand and gravel for use in the drain and transition zones and riprap bedding of the dam and dyke will be obtained from deposits B50, B100, B530, B540, B550 and B620. Concrete aggregates will be processed from deposit B50 upstream of the site or from B530 or B540 on either side of the access road to the project site. The location of these deposits and an indication of the volume of material available are shown on Plate 15. A brief description of the material found in each deposit follows.

Deposit B50

This deposit comprises two terraces located south of the Bitango Bridge. Of the three terraces at this location, the upper and middle ones contain gravel. There is a gravel pit and a crushed gravel stockpile on the upper terrace. The material is well graded to poorly graded sand and gravel (GW to GP)⁽¹⁾ with cobbles, boulders, and minor sand beds (SP). The oversize material comprises 5 to 25 percent cobbles between 150 and 200 mm diameter, and up to 10 percent boulders to 300 mm diameter. Some gravel and cobbles are coated with calcium carbonate. The deposit contains up to 5 percent platy, and up to 1 percent elongated, sandstone gravel. There is a trace of coal fragments and ironstone concretions.

(1) Unified Soil Classification Symbol.

TABLE 4-6
GLACIAL TILL - TEST RESULTS SUMMARY

TEST	LEFT BANK	RIGHT BANK
In situ moisture content		
Average	20	14.5
Range	8.5 - 26.6	9.2 - 18.4
Atterberg Limits		
Liquid Limit	45	46
Plastic Limit	19	18
Plasticity Index	26	28
Grain Size Distribution (%)		
Clay	38	37
Silt	43	45
Sand	15	15
Gravel	4	3
Standard Proctor Compaction		
Max. Dry Density (kg/m^3)	1,778	1,758
Optimum Moisture Content	17.5	17.3
Compression Index		
Virgin curve	0.19	0.17
Rebound curve	0.06	0.06
Effective cohesion (kPa)	33.9	21.6
Effective Angle of Shearing		
Resistance	23.5°	22.2°
Undrained Shear Strength (kPa)	196	178
Specific Gravity	2.74	2.74
Dispersion	ND1	ND1
In situ density (kg/m^3)	2080	2080
Permeability (cm/s)	4×10^{-7}	2×10^{-7}

Deposit B100

This deposit is the closest alluvial deposit upstream of the dam. It comprises well graded to poorly graded sand and gravel (GW to GP), with some cobbles and boulders, and minor, local, silty sand and gravel (GM) and sand (SP). The oversize material comprises 10 to 35 percent cobbles to 200 mm diameter, and up to 30 percent boulders up to 350 mm diameter. Some gravel and cobbles are coated with calcium carbonate. The deposit contains up to 15 percent platy and up to 1 percent elongated sandstone gravel. It contains chert fragments, ironstone concretions, and a trace of rootlets.

Deposit B530

This deposit is a terrace on the south side of the valley. Coho Paving Ltd. operates a gravel pit in this deposit. The material is well graded to poorly graded sand and gravel (GW, GP) with an estimated 2 to 5 percent cobbles. Gravel and cobbles are coated with calcium carbonate. Between 5 and 10 percent of the gravel and cobbles are platy or elongated. There is a trace of ironstone concretions.

Deposit 540

This deposit is located southeast of Deposit B530. It comprises sand and gravel (GP) with 2 to 5 percent cobbles and boulders to 300 mm diameter. The gravel and cobble sizes have some calcium carbonate coating, and include some ironstone concretions.

Deposit B550

This deposit is an outwash or delta, and is located in the upland of the river, and to the south of deposits B530 and B540. It comprises poorly graded sand and gravel (GW, GP) with 25 to 35 percent cobbles up to 200 mm diameter, and 10 to 15 percent boulders up to 300 mm diameter.

Deposit B620

This alluvial gravel deposit is located downstream of the spillway. It contains well-graded to poorly graded sand and gravel (GW, GP) with 25 to 35 percent cobbles up to 200 mm diameter and 10 to 15 percent boulders up to 300 mm diameter.

4.12.4 Riprap

Although the Buffalo Jump prospect has obvious haul advantages, durability of the sandstone is questionable. The laboratory tests show significant breakdown under freeze-thaw conditions and chemical attack. The abrasion resistance is just acceptable. Reconnaissance of similar sandstone outcrops in the vicinity of the project site has indicated no better quality rock. Tests on samples from the Blairmore prospect and the south quarry at Summit Lime Works indicate that these prospects will be more durable than the Buffalo Jump. It has been assumed, for the purposes of developing the capital cost estimate, that riprap will be obtained from the Blairmore area. The location of the riprap sources are shown on Plate 16.

An analysis was done to determine what seismic ground motions should be allowed for in design of the project structures. Three different approaches were used.

A probabilistic study was performed by officers of the Earth Physics Branch of Energy, Mines and Resources Canada, using the most complete data files available on past earthquake occurrences in Canada and surrounding regions. The method is known as Cornell-McGuire Seismic Risk analysis. It results in an estimate of the probability of exceeding various levels of seismic intensity at the site (expressed in terms of ground acceleration and velocity) during the service life of the structures. Based on the record of seismic activity in the past, this shows that there is about 5 percent probability that peak horizontal ground acceleration of 5 percent g may occur in a 50-year service life. This would be equivalent to a sustained horizontal acceleration of 3.3 percent g.

The second analysis, performed in-house, selected a maximum credible ground motion. This is the seismic intensity which will not be exceeded at the site at any time, without regard to its probability of occurrence. This is done by selecting a maximum magnitude earthquake for each of the zones outlined by the Earth Physics Branch (for their derivation of the Earthquake Zoning maps for the new National Building Code). These maximum magnitudes are generally about one-half unit of magnitude larger than any reported event in the zone. These hypothetical events are then assumed to occur on the boundary of the zone (the boundaries were also determined by E.P.B.)

at the point nearest to the site. With the given magnitude and distance from the site, the approximate expected intensity at the site can be calculated, using an equation developed for Western Canada and generally accepted for such use. The resulting maximum credible ground motion at the site is characterized by a peak horizontal acceleration of 13 percent g. This is derived from a maximum credible earthquake (MCE) of magnitude 6.5 at a distance of 50 km, on the eastern edge of the southeastern B.C. seismic zone, and at a focal depth of 10 km.

Both foregoing analyses assumed that there will be a continuation in future, of the pattern of seismic activity recorded in the past. The third approach was to consider the possibility that human activity might cause changes in that pattern. It was noted that there is a spacial correlation between some minor earthquakes and some mining and hydrocarbon extraction activities in Alberta and adjacent areas (in the vicinity of Crowsnest, Kimberly, Turner Valley, and Rocky Mountain House, for example). No such correlation is evident for the Pincher Creek area.

Existing reservoirs in similar tectonic situations were also examined to determine whether any had triggered detectable earthquakes. No such correlation was found, even with reservoirs much larger than the one to be impounded on Oldman River, such as the reservoirs of Libby, Hungry Horse, Arrow, Duncan, Bennett, and Mica dams.

It was concluded from this study that earthquakes triggered by the Oldman River reservoir are extremely improbable and would not, if they should occur, cause ground motions larger than have been anticipated in design. However, minor earthquakes triggered by future mining or hydrocarbon operations cannot be entirely excluded, and a seismic monitoring program is recommended.

SECTION 5

PROJECT LAYOUT

5.1 LAYOUT STUDIES

Alternative arrangements of the major structures were studied to determine the optimum project layout with respect to technical performance and cost.

A brief review of possible alternative sites in the immediate vicinity of the project location, considered in previous studies, did not reveal other sites which merited further consideration.

Layout sketches are presented on Plate 17 for alternative spillway and tunnel arrangements located on the right and left banks. These represent the optimum arrangement for three basic combinations of spillway and tunnel location. The fourth, with the spillway on the left bank and the tunnel on the right bank, is the least attractive and is not shown on Plate 17. The relative merits of these arrangements are discussed below.

5.2 SELECTED PROJECT LAYOUT

The layout of the major project structures adopted for the project is shown as Layout A on Plate 17 and is detailed on Plate 18.

5.2.1 Right Bank Spillway

The flip bucket location and chute orientation have been chosen so as to align the discharge in the direction of the river valley downstream. This will minimize flow eddies around the downstream toe of the dam and the flip bucket structure during spillway discharge. The approach channel alignment shown is the optimum with respect to the volume of excavated materials that can be used in the dam. There would be a slight reduction in excavation quantities if the entrance of the channel was moved to the east but such an alignment would introduce undesirable hydraulic conditions in the tightly curved approach channel.

5.2.2 Dam

The dam has been positioned to minimize the fill quantities taking into account the need to minimize the length of the tunnels around the dam and to minimize the spillway chute excavation. The latter has been achieved by placing the headworks as far downstream as the interface between the dam and spillway will allow. The downstream limit of the dam is governed by the need for an appropriate separation between the toe of the dam and the plunge pool to minimize the possibility of toe erosion by back eddies during spillway discharge. The tunnel intake has been located as far downstream as possible, to found as much of the dam as possible on the competent sandstone that outcrops at the bedrock surface at the downstream end of the valley. Weaker mudrocks predominate at the upstream end. The right side of the dam has been located to place the core of the dam within the deeply incised gully on the right bank.

A good balance between the volume of excavated materials and their use in the dam has been achieved with Layout A. This will be refined during final design.

5.2.3 Left Bank Tunnels

The intake portal has also been located as far downstream as possible to minimize tunnel length and because the rock conditions for the portal appear to improve in the downstream direction. The outlet portal location has been selected for similar reasons and also to provide an appropriate site for a possible future powerhouse. It has been positioned to avoid possible damage to the low level outlets and powerhouse from flip bucket discharges.

5.3 ALTERNATIVE SPILLWAY LAYOUTS

5.3.1 Left Bank Spillway

This alternative arrangement is shown as Layout B on Plate 17. The position of the flip bucket has been chosen so as to achieve a substantial separation between the downstream toe of the dam and the point of discharge of the spillway. As the alignment of the spillway discharge is almost perpendicular to the river valley, significant eddying and possible erosion of the toe of the dam could occur. The gate structure would be founded on bedrock and the spillway chute would be founded on till for the upstream portion (20 horizontal to 1 vertical) and on bedrock for the downstream portion (3 horizontal to 1 vertical). The diversion tunnels would also be located on the left bank with this layout alternative. The dam would be founded on the downstream sandstone outcrops and in a position requiring the least volume of fill.

There are significant cost savings for Layout A with the right bank spillway as compared with Layout B with the left bank spillway location. Also, since the functional aspects of the right bank spillway are superior, because the discharge can be directed along the river valley, and since the left bank spillway chute foundation design would be complicated by the dissimilar foundation conditions along its length, there is a clear advantage in favour of Layout A.

5.3.2 Right Bank Tunnel Layout

Layouts C and D on Plate 17 show possible locations for the diversion tunnels on the right bank between the spillway and the dam. Layout C shows the diversion tunnels with no provision for a powerhouse. Layout D is based on provision of the diversion tunnels and a powerhouse. In both instances, the dam has been moved upstream from the position shown in Layout A, to provide space for the spillway and the tunnel outlet portals.

A comparison of these layouts with Layout A, shows that, although there is a decrease in the length of the tunnels, there is a corresponding increase in both the length of the spillway and the volume of dam fill. The volume of the dam for Layout D would be greater than that for Layout C. The overall effect is that both Layouts C and D would cost significantly more than Layout A.

If the space requirement for the tunnels is met by moving the spillway downstream instead of moving the dam upstream, the spillway chute length would again be increased and the excavation quantity at the downstream end of the chute would become larger.

SECTION 6

RIVER DIVERSION

6.1 DIVERSION CONCEPT

During the three years required to complete construction of the Oldman River dam, river flows will be diverted by cofferdams into two tunnels excavated in the left abutment as shown on Plates 18 and 19. The cofferdams will be constructed across the full valley width upstream and downstream of the dam so that dam foundation excavation and preparation and dam fill placement can be carried out in the dry.

The upstream cofferdam will be incorporated into the dam embankment. The downstream cofferdam will be located to serve both the dam embankment and the spillway construction. It will be removed prior to commissioning.

The diversion waterway will be two parallel, concrete-lined, tunnels with common reinforced concrete intake and outlet structures. The two circular tunnels will have an inside diameter, after lining, of 6.5 m. The centrelines of the tunnels will be 16 m apart. The tunnel lengths average 900 m.

6.2 ALTERNATIVE ARRANGEMENT

A brief study was carried out concerning the employment of a concrete conduit in the river valley to carry construction diversion and permanent discharges. The comparative costs developed for this study indicated that no cost benefit would be obtained with this form of diversion when compared to the use of the two tunnels. The possible hazard

associated with cracking of the conduit due to foundation and dam fill deformation and consequent seepage and piping along the contact with the dam core also was considered. It was concluded that construction diversion should be accomplished using twin tunnels.

6.3 DESIGN FLOOD

Annual flood flow peaks on the Oldman River occur in May and June. It is common practice to set the cofferdam height and tunnel capacity to divert floods in the 10 to 50 year return period range. The choice of flood used for design of the diversion facilities is a compromise between economy and acceptance of the risk of downstream damage.

The diversion scheme for the project calls for the upstream cofferdam to be constructed to pass the 1:50 year flood through the tunnels. A higher cofferdam would allow a larger flood to be passed with an attendant lower risk of overtopping but greater potential for downstream damage if overtopping occurs.

As the major upstream cofferdam works will become part of the upstream toe of the dam, as shown on Plate 24, they should not enter into the economics of the decision. More important is the quantity of fill that can be placed in the cofferdam section between flood seasons. Tunnel size and length are the major economic factors to be considered.

There are two important downstream structures which would be jeopardized by cofferdam failure. They are Summerview bridge 6 km downstream of the site and the LNID (Lethbridge Northern Irrigation District) diversion headworks located some 40 km

downstream of the site. There are other bridges available if the Summerview bridge were to be damaged, but overtopping of the earthworks at the LNID headworks could result in considerable cost and construction problems for replacement of the works, and in potential loss of an irrigation crop year.

Based on the foregoing considerations, downstream flood damage has been assessed and related to the cost of the diversion works.

6.4 FLOOD RISK STUDIES

The effect of overtopping of a cofferdam on river flows downstream of the project site was investigated using the Dam Break Flood Forecasting Model developed by the U.S. National Weather Service (NWS).

The results of NWS model analysis are summarized on Plate 20. Overtopping of cofferdams with a crest elevation of 1081.5 m could generate a flood wave that would overtop but not necessarily damage the Summerview bridge. It would not exceed the flood capacity of the LNID headworks. During final design this analysis will be refined and the cofferdam height set so that overtopping will be unlikely to occur.

6.5 DIVERSION SEQUENCE

The diversion sequence selected is as follows. The diversion flood hydrographs are shown on Plate 21. The diversion phasing is shown schematically on Plate 22 and the construction schedule is shown on Plate 32.

Diversion will commence after the 1988 spring flood has passed and the starter cofferdam has been placed across the river to elevation 1060 m and the cofferdam protecting the tunnel intake portal has been removed. The river will be diverted through both 6.5 m diameter lined tunnels. Construction of the dam will then proceed with the upstream shell being placed as the upstream cofferdam. The risk of overtopping will decrease as the upstream cofferdam is raised. Table 6-1 shows the elevation to which the cofferdam will be constructed during the flood flow period of subsequent years and gives the corresponding frequency of the largest flood that can be passed without risk of overtopping the cofferdam. The table also presents hydraulic data.

TABLE 6-1
DIVERSION FLOOD FLOOD DATA

SPRING	COFFERDAM ELEVATION	FLOOD FREQUENCY	FLOOD INFLOW	TUNNEL OUTFLOW
	(m)	(yr)	(m ³ /s)	(m ³ /s)
1989	1080	1:50	1560	1000
1990	1100	1:1000	2970	1260
1991	Dam completed to crest elevation 1125.5 m.			

INTAKE

The reinforced concrete intake structure will provide an hydraulically efficient entrance to the tunnels. Two converging wingwalls will serve to direct flows into the twin intakes. The walls will include guides into which stoplogs can be inserted for dewatering the tunnels. A single set of intake stoplogs and follower will be provided. These will be handled by crane. After the tunnels have completed their diversion function, trashracks will be installed in these guides for permanent trash protection.

Rounded transitions, will be provided where the headwall abuts the intake to minimize the formation of air core vortices during full tunnel flow. The headwall and wingwalls will also act as retaining walls for fill materials placed to stabilize the near vertical excavated rock slope above the portal.

A rectangular-shaped intake with a bellmouth roof will maintain positive pressures on the intake roof at design discharge and during subsequent riparian and irrigation flows. This rectangular-to-square section will be followed by a short, square to circular transition to the tunnel.

Open excavation above the intake portal will be to the slopes shown on Plate 19. The overall slope in rock will be 1.7 horizontal to 1 vertical. This will be achieved by excavating the rock almost vertically above the portal and above this, the slope will be 1.5 horizontal to 1 vertical with 15 m wide horizontal benches generally at 20 m intervals of height. A 10 m wide bench section will be formed at the contact of rock and overburden. Slopes of 2 horizontal to 1

vertical will be used in the glacial till above FSL and 3 horizontal to 1 vertical below FSL. Similarly the excavated slopes in the overlying glacio-lacustrine silt and clay will be 3 horizontal to 1 vertical and 4 horizontal to 1 vertical respectively. Benches wider than 10 m will be required for construction purposes in some areas.

6.7 OUTLET

The outlet structure will be designed to provide a smooth and efficient exit from the two tunnels. Each tunnel will have a round-to-rectangular transition section designed to conserve head and increase capacity. This transition will be followed by a rectangular box section that provides access into the tunnels for maintenance and a downstream stoplog section for dewatering. Stoplog guides will be provided in the concrete structure.

The tunnel outlets will be subjected to high velocity flows only during the diversion of high flood flows. The outlet structure will be protected against undermining by a vertical cut-off wall extending from the foundation of the outlet section into bedrock.

The excavated rock slopes above the outlet portal will be similar to those described for the intake portal in Section 6.6.

6.8 DIVERSION CLOSURE

Upon completion of the construction of the dam and spillway, the west tunnel will be dewatered by placing stoplogs at its upstream and downstream portals. This work will be carried out after the 1990 flood season during the period when a single tunnel can carry the total river flow. Once the tunnel is dewatered, work will commence on the installation of the tunnel plug, control valves, guard gate and associated concrete work. Upon completion, the stoplogs will be transferred to the second tunnel where similar work will be completed. The stoplogs will then be removed and stored. Reservoir filling can then take place.

6.9 TUNNEL DESIGN

6.9.1 Tunnel Sizes

The major cost of diversion is the cost of the tunnels and of the inlet and outlet portal excavations. However, the latter are effectively common to all tunnel sizes and so the smallest diameter, shortest tunnel with the highest possible cofferdam would therefore constitute the least costly diversion system. The location and length of the tunnels has been fixed as part of the overall project layout considerations. The crest of the cofferdam, during the first year of dam construction, has been set at a maximum elevation of 1080 m to avoid damage to the LNID headworks if the cofferdam should be overtopped.

A tunnel diameter of 6.5 m for each of the two tunnels was selected as appropriate to the cofferdam height, for the following reasons.

- The combination of tunnels and upstream cofferdam will carry the routed 1:50 year annual flood following the first year of dam construction.
- The maximum tunnel velocities of 15 m/s are acceptable for a concrete lined tunnel.
- The hydraulic conditions will be stable at design flows.
- The size is suitable for a range of tunnel boring machines.

Hydraulic stability and energy dissipation will be further investigated in the hydraulic model study.

6.9.2 Tunnel Shape

A circular tunnel section has been selected for the preliminary design. It has the following advantages over other tunnel shapes.

- The concrete liner is a simple structure requiring less concrete and reinforcement than other shapes.
- It is appropriate if a tunnel boring machine is used for excavation.

It has two disadvantages.

- The circular floor is less convenient for construction traffic.

- It is less appropriate for the drill-and-blast method of excavation which could, if not blasted carefully, result in an irregularly shaped excavated section in the horizontally bedded and vertically jointed rock, requiring more concrete to fill the overbreak, than for a horseshoe or inverted "U" shaped tunnel.

6.9.3 Hydraulic Design

The invert of the inlet will be at elevation 1048 m and that of the outlet will be at elevation 1043 m. These elevations will ensure submerged flow through the tunnels at the design discharge and open channel flow at lesser discharges. Under the 1:50 year flood conditions, the upstream water level will be at elevation 1080 m. This will result in water velocities in the tunnels of about 15 m/s. The velocity will increase to 18 m/s for the extreme 1:1000 year diversion flood peak inflow of $2970 \text{ m}^3/\text{s}$. Considering the flood probability, these velocities are acceptable for a concrete lined tunnel for the short period that such flows would occur.

6.9.4 Geotechnical Design

The diversion tunnels will be driven through the left bank in an interbedded sequence of predominantly siltstones and claystones with some sandstones. The bedding is horizontal and individual beds are less than 3 m in thickness.

The tunnels will be driven in a low stress environment. The total vertical rock load is modest. No measurements have been made of rock stresses in the vicinity of the tunnel but the open joint systems observed in test trenches and in test holes near valley slopes indicate a large measure of

stress-relief. It is possible that the ratio of horizontal to vertical in situ stresses (K_0) may be greater than unity. Nonetheless, rock stresses surrounding the tunnel are considered to be much less than the overall strength of the rock mass surrounding the tunnel. Thus no major problems of stress-induced rock failure are anticipated.

Maximum ground cover will be 70 m and the hydrostatic head of groundwater during tunnelling is likely to be less than 5 m. The separation of the tunnels by two tunnel diameters (centreline to centreline) will preclude interaction between the tunnels.

Deformation produced on excavation by the readjustment of the stress field surrounding the tunnels will also occur. This has been assessed and roof support measures will be designed to control the effects of that movement where necessary. The design of support for underground excavations requires that the deformations induced by the dead weight of loosened rock as well as those induced by a readjustment of the stress field surrounding the excavation must be taken into account.

Modelling of the geotechnical response of the rock and lining to tunnel excavation will also take into account the operational hydraulic load (equivalent to full supply level) in the section upstream of the valve, as well as the high internal pressures which would be developed during the operation of the tunnel if the power generation option is chosen.

There is no evidence to date that would indicate that the materials at the site exhibit significant time dependent behaviour other than mechanical deterioration or slaking. The preliminary design considerations have therefore been restricted to a combination of linear-elastic and plastic behaviour around a circular excavation. Structurally controlled failures are likely from the haunches of the tunnel where intersections of dominant sub-vertical jointing and horizontal bedding will cause blocks to be loosened. These will be analysed during final design and will require specific assessment on site for stabilization.

6.9.5 Tunnel Support Analysis

The rock-support interaction analysis developed by Hoek and Brown (1980) was used to generate the curves shown on Plate 23 using the input parameters given in Table 6-2. The curves show the ground reaction of the rock mass as an initial linear-elastic response followed by a plastic response. This models the requirements of interactive support with deformation subsequent to excavation. The graphs of straight line support indicate the stiffness of the support system and the maximum pressure which the support can sustain before yielding.

Two cases were analysed. The "worst case" modelled a fair quality rock mass with moderately weathered joints spaced at 0.3 to 1 m and accounted for full reservoir water pressure. This case could occur near the inlet portal. The "typical case" modelled a very good quality rock mass with tight unweathered joints without hydraulic pressures. This case would occur in the harder, undisturbed rocks downstream of the valve chamber.

It is apparent from the results of the analysis that in the typical case, the rock response would be almost completely in the elastic range and would not require support to inhibit stress-induced deformations. By comparison the worst case would require initial support for stabilization, prior to the installation of final support in the form of the tunnel lining. The test tunnel program, described in Section 4, has been designed to provide data to confirm these results.

Based upon this analysis, initial and final tunnel support systems were selected. The initial lining will consist of shotcrete and bolts. The final lining will be cast-in-place concrete.

6.9.6 Initial Tunnel Support

Based on the rock-support interaction analysis of stress-induced deformation, and consideration of potential structural failures which might develop the need to support the weight of loosened rock above the tunnel, the following tunnel support has been used for preliminary design and is shown on Plate 19.

- A 2 m x 2 m pattern of 4 m long Swellex bolts as part of the regular tunnel advance within 3.5 m of the face.
- Spot bolting, as necessary, using tensioned resin-anchored bolts to stabilize local structurally controlled failures.
- A 50 mm thick application of shotcrete within 10 m of the advancing face, possibly to invert level, but generally over 75 percent of the tunnel perimeter.

TABLE 6-2
ROCK-SUPPORT INTERACTION ANALYSIS
INPUT PARAMETERS

	WORST CASE	TYPICAL CASE
<u>Rock</u>		
In Situ Stress (MPa)	2.44	1.5
Intact Rock Uniaxial Strength (MPa)	3.21	30.0
Original Rock Mass, m	1.38	3.4
Original Rock Mass, s	0.002	0.008
Elastic Modulus (MPa)	1000	5000
Poisson's Ratio	0.5	0.25
Broken Rock Mass, Mr	0.18	1.38
Broken Rock Mass, Sr	0.0001	0.002
Unit Weight, Broken (MN/m ³)	0.02	0.02
Mr, Improved	0.82	0.82
Sr, Improved	0.0003	0.0003
<u>Concrete</u>		
Elastic Modulus, Concrete (MPa)	21,000	21,000
Poisson's Ratio	0.25	0.25
Uniaxial Strength (MPa)	35	35

Primary shotcrete, bolts and dowels will need to be provided as close to the face as possible. Some rib support may be needed near the portals. Mechanical anchorages are inappropriate for the rock conditions found at the site and are not being considered for the tunnels. The primary support will be installed as the tunnel excavation progresses.

6.9.7 Permanent Lining

The permanent concrete lining will range from a thickness of 400 mm upstream of the valve chamber to a nominal 250 mm downstream of the valve chamber. The upstream lining will be reinforced. After placement of the permanent lining, low pressure grouting will be used to fill the voids between the concrete and the rock. Additional consolidation grouting, if required to prevent seepage, will be carried out after the concrete has reached design strength.

The structural design considerations for the permanent tunnel lining will include the following:

- Bending moments induced by long term rock deformation.
- Rock loadings.
- An external water pressure corresponding to the groundwater conditions upstream of the valve with the reservoir level at FSL elevation 1,118.6 m. Should the tunnels need to be dewatered in the future, the reservoir must be drawn down by releasing through the service spillway prior to upstream stoplog installation using a barge and divers.

- A different pressure head that could develop in the downstream portion of the tunnel where negative pressures may materialize during high velocity diversion flows.
- An internal water pressure corresponding to a reservoir level at FSL elevation 1,118.6 m, plus an allowance for hydraulic transient pressures induced by rapid shutdown of a possible future powerplant.

6.9.8 Instrumentation

The portals and tunnels will be instrumented and monitored to determine post-excavation movements. Instruments will be installed at selected locations immediately after excavation to measure elastic deformation behaviour. Readings will be taken frequently after excavation and the frequency will be gradually decreased until the permanent works are in place.

Observation of formation movements within the tunnels and at the portals are required:

- to confirm that initial tunnel support systems and design assumptions are appropriate, and
- to identify potential failures in portal slopes and permit timely remedial works.

Multiple point rod extensometers installed in drillholes are proposed at the following locations.

- A row of five extensometer arrays across portal excavation slopes at about 2 to 4 m above tunnel crown with anchors at each array set 2, 5 and 15 m into the slope.

- Three arrays of extensometers at the tunnel springline and crown at 4 locations in each tunnel, 20 m and 300 m in from each portal. The anchors will be set at 2, 5 and 15 m depths.
- One array of extensometers at the valve chamber excavation at the springline and crown with anchors set at 2, 5 and 20 m depths.

SECTION 7

LOW LEVEL OUTLETS

7.1 GENERAL ARRANGEMENT

Both diversion tunnels will be equipped with a control valve, a guard gate and a valve controlled, small diameter riparian outlet set below the winter ice level. Each control valve will be sized to individually pass the maximum required reservoir releases. If there is a possibility that a powerplant could be added at a future date, the installation in the east tunnel will also include the minimum provisions to allow for a future connection to a penstock. These are shown on Plate 34 and described in Section 14.

The tunnels will be plugged with concrete at approximately the midpoint of the tunnel length. A valve chamber will be excavated above the tunnel plugs for housing the valve operating mechanisms, maintenance, crane and other mechanical and electrical equipment for each tunnel. Access to the valve chamber will be from the left bank drainage tunnel. The chamber will be connected to an air shaft to the surface. This air shaft will also provide emergency access to the valve chamber. The valve chamber and access shaft are shown on Plate No. 19.

The single set of stoplogs with follower used for river diversion will be designed to permit tunnel dewatering after the reservoir has been impounded. This will allow access to the upstream portion of the tunnels if required for maintenance of the tunnels or guard gates. This is not anticipated to be a routine maintenance activity, and

placement and removal of stoplogs will require a crane on a barge, with divers. A combination of lowering the reservoir level and specialized diving equipment will be required. Scuba diving with normal compressed air breathing apparatus, is not possible to the depths of water involved. The use of mixed air apparatus and provisions for decompression will be necessary. The operation will require very careful planning in conjunction with diving specialists. The stoplogs and guides will be designed to ensure that the components can be easily manipulated in deep water. Both tunnel intakes will be provided with trashracks which will be located in the same guides as used for the stoplogs. The trashracks will also be cleaned by divers.

Each tunnel outlet structure will have a rectangular chamber upstream of the stop logs. These chambers will provide access for men and equipment to carry out maintenance and construction work. A single set of stoplogs with follower will be provided for the outlet works. Each chamber will be provided with a sump and a rail mounted, level controlled, removable sump pump.

7.2 HYDRAULIC DESIGN CRITERIA

The low level outlet will discharge approximately $40 \text{ m}^3/\text{s}$ under minimum head with the reservoir at minimum supply level (MSL) of elevation 1064 m. It will discharge at higher flows with increasing head to a maximum of $80 \text{ m}^3/\text{s}$ at FSL. This will meet the preliminary design criterion that the design flow should be based on the extreme case of a complete and steady drawdown of the $490,000 \text{ dam}^3$ of live storage over a six month period.

The capability of the low level outlet to operate at its full capacity under high head conditions will be studied during model tests. The very high flow velocities that would exist at the vertical slide gate guides, may require that the valve flows be limited under high head conditions.

The riparian flow conduit will be capable of passing $10 \text{ m}^3/\text{s}$ at minimum reservoir level when the hollow cone valve is closed.

7.3 STRUCTURAL DESIGN CRITERIA

The tunnel plugs will be designed to take the full hydraulic pressure through shear keys located around the plug. These shear keys will be filled with a lean mix concrete during diversion to avoid risk of cavitation.

7.4 EQUIPMENT SELECTION

The 2 m diameter hollow cone valve (also known as Howell-Bunger valve) has been selected because of its high discharge coefficient, high energy dissipation and balanced design. In comparison with other types of outlet valves, the power required to drive the operating mechanism for the hollow cone valve is very small and the hydraulic unbalance forces during partial openings are very nominal. The provision of sufficient vented air has resulted in smooth operation of such valves in many installations. The valve has also proved satisfactory under partial and complete submergence. The cost of this type of valve for the design head on this project is also not significantly different from that of other types of valves. Since the valve is located at the centre of the tunnel length no icing problems are expected.

The equipment in the two tunnels will be identical.

The water passage in each concrete plug will consist of a bell-mouth inlet to a 2000 mm diameter steel liner. A transition piece will be provided between the circular steel liner and the guard gate. The guard gates will be custom designed 2 m diameter bonnetted ring follower slide gates, which will provide for upstream maintenance of the hollow cone valves and to any requirements for operation under non-balanced conditions. A manhole will be provided on top of the embedded steel liner upstream of the guard gate for access to the tunnel upstream of the concrete plug. A 600 mm diameter vertical shaft from the steel lined water passage will lead to a manhole in the valve chamber floor. A 100 mm diameter bypass line complete with a shutoff valve located in the valve chamber will be provided to balance the pressure across the guard gate.

A flange will be provided at the end of the steel liner onto which the 2000 mm diameter hollow cone valve will be bolted. It is anticipated that operation will be by hydraulic cylinder.

A 600 mm diameter pipe to provide riparian flow releases will be embedded in the concrete plug and routed through the valve chamber. A motor operated flow control valve and a shutoff valve will be provided on this line. The flow control valve will be operated remotely to adjust riparian flow when there are no other demands. The shutoff valve will be used to isolate the flow control valve for maintenance.

A 100 mm diameter pipe will be embedded in the concrete plug at the tunnel invert. A shutoff valve will be provided at the downstream end of the pipe and inside a concrete cut out. The valve will be opened manually to drain the tunnel below the cone valve invert when required for inspection.

A 2.1 m diameter ventilation shaft will connect the vertical access shaft to the discharge chamber downstream from the concrete plug. Detailed consideration will be given to venting above tailwater level, to avoid flooding of the valve chamber.

The concrete below the hollow cone will be sloped away from the concrete plug. The tunnel downstream of the plug will be steel lined at a distance of 12 m up to the location of a circumferential deflector ring around the tunnel.

The steel deflector ring will be provided above the valve to limit reverse flow of water on the roof. Inserts will be provided in the roof of the steel lining above the hollow cone valve to allow attachment of a monorail crane beam for dismantling of the valve.

The control room equipment will include a hydraulic power pack for supplying hydraulic oil under pressure to operate the guard gate hoists and the hollow cone valve. The hydraulic power pack will consist of two motor driven hydraulic oil pumps, an accumulator tank and all the necessary valves and controls. A monorail crane will be provided in the valve chamber for dismantling of hydraulic cylinders and the guard gates if necessary.

The equipment in the valve chamber including the gates can be removed through the vertical access shaft. An access way with watertight doors will be provided from the valve chamber to the tunnel section downstream of the concrete plug, which will allow inspection of the hollow cone valves under normal operating tailwater levels. The downstream side of the guard gate can also be inspected through the hollow cone valves. Dismantling and removal or replacement of the valve when the guard gate is closed will require draining of the tunnel between the plug and the outlet stoplogs.

Inspection of the tunnels upstream from the concrete plugs and maintenance of the guard gates will require draining of the tunnel by pumps at the tunnel outlet.

Trashrack cleaning will need to be carried out by divers at regularly scheduled intervals.

SECTION 8

DAM AND DYKE

8.1 ALTERNATIVE DAM TYPES

Both embankment and concrete gravity dams have been considered for the project. The merits of constructing a concrete gravity dam using roller compacted concrete as the basic construction medium were evaluated. This alternative was found to be more costly than the embankment dam.

8.2 DAM SECTION

The dam will be a zoned earth and rock fill structure as shown on Plate 24. The dam section has been designed to use the materials available from the required excavations to the maximum extent possible in a manner that will minimize and control seepage, and be stable under all loading conditions.

8.3 FOUNDATION

Within the core contact zone alluvium and colluvium will be removed prior to foundation treatment. The mudrock overlying the Basal Sandstone Sequence in the valley bottom is thin and there are indications that it has weathered and softened. It will also be removed within the core contact zone to expose competent rock. The shells will be founded on the alluvium and colluvium after the surface has been stripped to remove loose, high silt content, potentially liquefiable, and organic materials.

As soon as the core trench has been excavated to final grade it will be cleaned with low pressure water jets. All joints and local rock defects will be filled with slush grout and dental concrete. Prior to grouting and placement of the first layer of core material, slush grout, dental concrete, shotcrete and if necessary, mud slabs will be applied to the exposed rock surfaces in the core contact zone for protection against weathering and to produce a surface to receive the first layer of Zone 1 fill.

8.4 DAM ZONES

The dam section and the zones of the different materials from which the dam will be constructed are shown on Plate 24. It should be noted that Zone 6 is not a dam fill zone. It is shown in Plate 24 as the in situ alluvium and colluvium foundation of the dam.

The central core will be constructed of glacial till, Zone 1. The width of the core contact zone at bedrock foundation will be approximately equal to the head of water on the dam. The core will be protected on the downstream side with a sand filter, Zone 2, transitioning to a more pervious sand and gravel drain, Zone 3. Both of these zones will be continued along the base of the downstream shell to form a drainage blanket that will intercept foundation seepage and prevent the migration of the fine grained material from the dam foundation into the drains. The drainage blanket will exit at the toe of the dam. On the upstream side, the core will be protected with a layer of Zone 2.

On the upstream side of the core, the dam will utilize suitable spoil from the spillway excavation in Zones 4 and 4A. In Zone 4, the specifications will accept, most of the excavated rock, which will consist of excavated mudrock with a minor content of sandstone. Overburden from the spillway excavation will be placed in the less critical Zone 4A.

Pitrun sand and gravel, Zone 5, will be placed on the outer layer of the upstream shell to enhance drainage and stability during drawdown. During fill placement the more competent sandstone and siltstone, that will be less susceptible to weathering breakdown, will be routed to the outer portions of the dam shell. Riprap, Zone 8, on a gravel bedding layer, Zone 7, will protect the upstream face of the dam from the erosive action of waves.

The downstream shell will also be constructed of Zone 4 rock material from the required spillway excavation, with the less competent and more impervious rock and soil being placed in the central Zone 4A. A pitrun sand and gravel filter, Zone 5, will be placed in between the shell and the drain Zone 3 to prevent the fine grained material of Zones 4 and 4A from migrating into and clogging the drain. A thin layer of Zone 5 pitrun sand and gravel will be placed on the downstream face of the dam, to stabilize the surface.

A nominal freeboard of 1.3 m above the PMF surcharged reservoir level will be provided to set the crest elevation at 1125.5 m. The core will be extended to elevation 1124.5 m. The dam will be constructed above this nominal crest elevation to allow for settlement of the fill and the foundations. The 12 m wide crest will include a gravel surfaced public road.

8.4.1 Zone 1 - Impervious (Glacial Till)

Glacial till will be used as the impervious core material for the dam and cofferdam. It will be obtained from the required spillway excavation and the left bank borrow area upstream of the dyke shown on Plate 15. The surficial glacio-lacustrine silt and clay must be excavated from the borrow area before the underlying glacial till can be removed. The glacial till consists of an upper layer of highly plastic clay till underlain by medium plastic silty clay till. The test data show that the in situ moisture content of approximately 70 percent of the till ranges between 1 percent greater, and 4 percent less than the optimum moisture content. This is shown on Plate 10.

The insitu moisture content of 70 percent of the till available from the right bank spillway excavation is on average between 1 percent greater, and 3 percent less than the optimum moisture content. This is shown on Plate 9.

Zone 1 will be well graded and within the gradation envelope shown on Plate 24. Minimal processing will be required to remove the low cobble and boulder content. The placement moisture content will be specified to be between 2 percent greater than, and 2 percent less than optimum moisture content. This will achieve maximum deformability of the core with minimal cracking, consistent with adequate strength for trafficability and overall stability. Zone 1 material will be placed in layers not thicker than 250 mm and will be compacted with a 50 ton pneumatic tired roller or tamping foot roller.

8.4.2 Zone 1A - Impervious (Random)

The dyke and the upstream impervious layer of the cofferdams will be constructed of Zone 1A material which will be specified to include glacio-lacustrine silt and clay, glacial till and other fine grained soils obtained from any source. Zone 1A will also be used to provide initial cut-off through the river alluvium.

8.4.3 Zones 2 and 3 - Transition and Drain

These materials will be obtained from river alluvial or terrace deposits described in Section 4 and shown on Plate 15. The specified gradation limit curves are shown on Plate 24. Gap graded material will not be accepted. Crushing, washing and blending will be required to remove the fines and to meet the specified gradation. Zones 2 and 3 will be placed in 250 mm layers and will be compacted with a vibrating steel drum roller.

8.4.4 Zone 4 and 4A - Random

Zones 4 and 4A rock will be obtained from the spillway and tunnel excavations. Zone 4 rock may include claystone, siltstone and sandstone, oversize material from glacial till, river alluvium and terrace deposits. Zone 4 material will have a 1 m maximum size limitation. No processing will be required.

Zone 4A soil may include glacial till, glacio-lacustrine silt and clay, outwash sand, and gravel, river alluvium and terrace deposits, with no size or moisture content limitations except that the fine grained glacio-lacustrine silt and clay may require moisture conditioning on the fill to ensure minimal compaction.

Zone 4A will be placed in 300 mm layers and Zone 4 will be placed in 1 m layers. Zones 4 and 4A will be selectively placed so that the fill will be relatively homogenous with no significant voids left between large size rock particles. Zone 4A will be compacted with a vibratory steel drum roller. Zone 4 will be spread with a bulldozer blade and compacted with a vibrating roller and the systematic routing of the hauling and placing of equipment.

8.4.5 Zone 5 - Select Pitrun Sand and Gravel

Zone 5 will be obtained from river alluvial and terrace deposits shown on Plate 15. No processing will be required other than selective loading in the borrow pit. Oversize materials will be moved on the fill to the edges of the dam. Zone 5 will be placed in 300 mm layers and compacted with a vibratory steel drum roller.

8.4.6 Zone 7 - Riprap Bedding

Zone 7 will be obtained by scalping the coarse fraction from the river alluvial and terrace deposits shown on Plate 15. Processing will be required to remove oversize material. No compaction will be required.

8.4.7 Zone 8 - Riprap

It has been assumed that Zone 8 will be quarried limestone obtained from the Blairmore prospect shown on Plate 16. It will be placed with larger sized particles uniformly distributed through the layer with smaller sized particles filling the voids.

8.4.8 Dam Crest

The dam will be capped with a layer of Zone 5 underlain by a transition layer of Zone 2. The crest will be 12 m wide and will accommodate the proposed public highway across the project. Guard rails and possibly a concrete parapet wall will be provided.

8.5 COFFERDAMS

8.5.1 Upstream Dam Cofferdam

The upstream cofferdam that will be required to dewater the dam foundation and to protect the fill placement work will be formed by constructing the upstream shell of the dam in stages as shown on Plate 22.

During 1988 the upstream shell of the dam will be built to elevation 1080 m to form the cofferdam. First a starter cofferdam will be constructed to elevation 1054 m by dumping impervious spoil from the required excavations into a trench excavated through the river valley alluvium. After the spring flood has passed the starter cofferdam will be completed by dumping spoil across the river to divert the river flow into the first completed tunnel. With the low fall river flows diverted and the pervious river alluvium cutoff across most of the valley, a full cutoff can be constructed on the downstream side of the starter cofferdam by placing and compacting Zone 1 material in the dewatered core trench. The excavated river alluvium will be placed upstream and downstream of this core. As an alternative a slurry wall trench will be considered for this stage of the cofferdam.

Following completion of this cofferdam to elevation 1060 m the upstream portion of the dam will be completed to elevation 1080 m using random river alluvium excavated from the core trench of the dam and rock from the spillway excavation. The cofferdam will be built within the limits shown on Plate 22. An impervious blanket will be placed on the upstream face of the cofferdam shells and will connect to the core of the first stage cofferdam.

Placement of the upstream portion of the dam will continue to elevation 1100 m as shown on Plate 22, during the 1989 construction season. The upstream impervious blanket will be continued up the face of the dam. After the dam has been completed to above FSL elevation 1118.6 m, the upstream impervious blanket will be removed down to elevation 1060 m and replaced with the riprap bedding Zone 7 and riprap Zone 8.

8.5.2 Downstream Spillway and Dam Cofferdam

Only one downstream cofferdam will be used for both the dam and spillway located as shown on Plates 18 and 22. It will be removed at the end of construction.

It will be constructed of tunnel spoil and river alluvium dumped across the river with an impervious barrier of dumped glacial till on the downstream face. It will have a crest elevation of 1054 m.

8.5.3 Tunnel Portal Cofferdams

The intake and outlet tunnel cofferdams will be constructed of river alluvium and spoil from the portal excavations. They will have an upstream impervious blanket of dumped

glacial till. The crest elevations to provide protection against the 1:50 year flood are 1056 m and 1054 m for the upstream and downstream cofferdams respectively.

8.6 DYKE SECTION

The reservoir requires the construction of approximately 2.3 km of dyke to contain the reservoir on the left bank. The maximum height of the dykes will be 10 m, with the average height being 3.5 m. The dyke section is shown on Plate 25.

The soil stratigraphy on the left bank consists of a shallow mantle of topsoil underlain by glacio-lacustrine silt and clay, fine silty sand and glacial till. The glacio-lacustrine silt and clay and sand deposits are stratified and are locally pervious.

The main body of the dyke will be constructed from Zone 1A material. It will have upstream and downstream slopes of 4 horizontal to 1 vertical to accommodate the variable and lower quality Zone 1A material. The dyke will be founded on the overburden after the topsoil and vegetation has been removed. To reduce seepage through the pervious sandy layers in the dyke foundation, a shallow cut-off trench will be excavated to an impervious layer.

A blanket drain of pitrun sand and gravel Zone 5 will be provided beneath the downstream shell of the dyke. A layer of Zone 5 material will also protect the downstream slopes. Riprap protection will be placed on the upstream slope.

8.7 DAM ANALYSES

The dam and dyke sections have been analysed for stability and settlement. Input parameters were based on laboratory test data and published data for similar materials.

In the final design stage, a stress analysis of the dam will be carried out using the finite element method. The stress distribution and displacement in the dam and its foundation, stability of the dam and foundation, load transfer between core and shells, and safety against hydraulic fracturing and cracking of the core will be checked. The response of the dam to dynamic forces under seismic loading will also be analysed.

8.7.1 Foundation Strength

The lithology and structural condition of the bedrock foundation underlying the dam are discussed in Section 4. The existence and continuity of structural discontinuities or seams found in the claystone components of the bedrock have been established and an assessment of the roughness of the seams has been made. The inferred locations of these seams within the claystone units J10 to J11 overlying the Basal Sandstone Sequence and in the mudrock unit L interbedded in the Basal Sandstone Sequence are shown on Plates 24 and 27.

Geological interpretation indicates that these seams result predominantly from the stress-deformation history of the project site and are due to stress relaxation following valley rebound, rather than to the presence of weaker clay materials or a higher clay content than found in the surrounding rock. This is borne out by the results of the

direct shear box tests on specimens of seams and unsheared claystone from drill core and from the core samples taken from the walls of the shaft. These showed that the silty clay and clay materials in seams have the same residual strength as adjacent claystones of similar mineral composition. As a consequence of the shear deformation that has occurred in situ, the seams exhibit considerably lower peak strengths and higher natural moisture content than in the adjacent unsheared claystone.

The direct shear test results have been summarized for different bedrock types on Plate 26. The results of direct shear tests on unsheared bedded claystone and on seams in the upper claystone layer units J10 and J11 and those in unit L are shown. A comparison between the index properties of the claystone and seam infillings indicates that the seam infillings are very similar to material in the adjacent claystone, with the exception of the water content. The higher water content and lower peak strength in the seam material are an indication of dilation and preshearing.

The results shown on Plate 26 give a residual effective angle of shearing resistance equal to 16° for the seams in the claystone units J10 and J11. A roughness angle of 1.5° has been assumed as discussed in Section 4. Thus, the operative angle of shearing resistance for these seams has been assumed to be 17.5° .

Plate 26 also shows the direct shear test results on seams from the mudrock layers of Unit L. The residual angle of shearing resistance is 12° for the unit L seams and the roughness angle is zero.

8.7.2 Stability Analysis

The stability of the dam will be governed by the characteristics of the fill, the alluvium and colluvium beneath the dam and the bedrock foundation.

Stability analyses were performed by computer using the modified Bishop method, and modified Janbu method and were checked using the Morgenstern and Price method. These analyses are all based on the theory of limit equilibrium.

The stability analyses have been performed considering circular failure planes entirely within the dam as well as composite failure surfaces extending into the bedrock. Analyses generally indicate that minimum factors of safety occur when the failure surface extends down to and through the seams in the upper part of the bedrock.

The foundation bedrock model shown on Plate 27 was used in the analyses. It includes the seams existing in units J10, J11 and L and assumes them to be continuous over the full length of each potential failure surface. The stability analyses have shown that the minimum factor of safety occurs along failure surfaces passing through the highest seam. Even through a failure surface passing through a seam in unit L mobilizes a lower operative angle of shearing resistance, the safety factor is higher due to the additional normal load and increased cross bedding shear resistance.

The stability analyses of the dam were carried out using the effective strength parameters shown on Plate 27.

The seismicity of the site is described in Section 4. The maximum credible earthquake (MCE) was adopted for the design and a horizontal acceleration of 0.065 g was used for the pseudo-static seismic analyses.

The results of the stability analyses carried out on the dam, and the critical potential failure surfaces are shown on Plate 27.

The safety factors computed are close to the required minimum values. This is considered to be satisfactory for preliminary design bearing in mind the conservative but realistic choice of dam and foundation parameters used in the analyses. The downstream portion of the dam in particular, will be more stable than the analyses indicate as the dam is assumed to be entirely founded on the claystone of the Upper Mudrock Sequence containing seams. In reality it is known that the claystone is thin over most of this part of the river valley and analysis of the drillhole and other data shows that it is to be absent over approximately 20 percent of the dam foundation where sandstone exists at bedrock surface. Also the two dimensional analyses do not take into account the stabilizing effect of the narrow valley. These and other such factors will be taken into account during final design.

Conventional practice in assessing the stability of dams under seismic loading is to assume, for preliminary shaping of the dam section, a pseudo-static sustained horizontal acceleration of around 70 percent of the peak MCE horizontal acceleration. The response of the resulting dam section to the dynamic forces of the peak MCE horizontal acceleration is then checked using a dynamic analysis. This analysis is

usually deferred until the final design stage. This approach is generally satisfactory, particularly for conventional dam sections with steeper slopes on competent foundations, as the assumption of a pseudo-static force gives a good indication of the performance of the dam under dynamic loading conditions. However, the limit equilibrium stability analysis of the dam section shown on Plate 24 indicates that the dam would be unstable under pseudo-static seismic loading of 70 percent of the MCE peak horizontal acceleration. This result is a function of the predominant effect of the strength of the seams on the dam design rather than a true indication of instability and undesirable response to dynamic loading. The false conclusion arises because sustained horizontal the pseudo-static force implied in the analysis is unrealistically large as it is derived from the large mass required for stability under static loading. It was therefore considered more realistic to assume a pseudo-static horizontal acceleration of 50 percent of the peak MCE horizontal acceleration for the preliminary design of the dam. A dynamic analysis will be used to check this assumption during the early stages of final design.

8.8 DYKE STABILITY

Stability analyses for the dyke were carried out using Bishop and Morgenstern stability charts. The assumptions made for the use of these charts are as follows.

- The dyke will have a maximum height of 10 m.
- Unit weight of the compacted Zone 1A fill is 21.6 kN/m^3 .
- Effective angle of shearing resistance of the compacted Zone 1A fill is 24° .

The factors of safety obtained for various loading conditions are shown on Table 8-1.

TABLE 8-1
DYKE STABILITY ANALYSIS

LOADING CONDITON	SLOPE	ASSUMED r_u	FACTOR OF SAFETY
End of Construction	U/S	0.3	1.55
	D/S	0.3	1.55
Steady State Seepage	D/S	0.2	1.75
Rapid Drawdown	U/S	0.4	1.35

8.9

DAM SETTLEMENT

Settlement of the dam will originate in the dam fill, in the overburden foundation and in the bedrock. A significant portion of the settlement within the dam fill and the overburden foundation will occur during construction. The balance, as well as the consolidation settlement of the claystone portion of the bedrock, will occur over a longer period of time and will require a settlement allowance to be included in the dam crest. An allowance is also required for the settlement due to seismic effects. For the preliminary design, a 1 m camber has been allowed for the combined consolidation settlement of the dam and foundation, and the seismic induced settlement. The magnitude of the seismic earthquake induced settlement of the dam will be determined more accurately during final design using the dynamic analysis.

8.10 ABUTMENT STABILITY AND EXCAVATION

The water table in the valley is between elevation 1,060 m and river level which is generally close to elevation 1,050 m. The river valley slopes within the project site are dry and are presently stable. Sloughing is limited to areas where river action or weather processes on the weaker, more friable zones leads to local oversteepening or undercutting. A few shallow-seated areas of instability exist in colluvium in the right bank.

The existing river banks have an average slope of 2 horizontal to 1 vertical on the right bank and 1.3 horizontal to 1 vertical on the left bank. Both slopes are covered with colluvium. Reservoir impoundment will result in saturating this colluvium layer and, during rapid drawdown of the reservoir, the surficial layer of colluvium will slide at the contact boundary between soil and underlying bedrock. Stability analyses show that the existing bedrock slopes on the right bank are stable and deep seated failures are unlikely to happen during reservoir operation. On the left side, analysis indicates that the existing slopes will be marginally stable during reservoir operation against a deep seated movement. However, local shallow sloughing of both bedrock slopes is expected.

8.11 WAVE ACTION AND FREEBOARD

The action of wind on the Oldman River Reservoir will produce wind setup, waves and wave runoff. The crest elevation of the dam and dyke has been established at 6.9 m above FSL to permit the reservoir to surcharge during extreme floods with a nominal 1.3 m freeboard, as discussed in Section 9. The maximum fetch has been calculated as 3.4 km and 9.4 km for

the dam and dyke respectively. The corresponding 1:100 year overwater wind velocities were estimated as 160 km/h and 190 km/h giving wave run-ups of 2 m on the dam and 3.5 m on the dyke respectively. Wind setup was estimated as 0.15 m. Hence, with the dam crest elevation at 1125.5 m, an acceptable margin is available above that required as a defence against wave action during normal operating conditions when the reservoir is at FSL.

8.12 SLOPE PROTECTION

The upstream slope of the dam will be protected by placing a 1.5 m thick layer of riprap with a D_{50} size of 700 mm from the crest down to elevation 1,060 m, which is 4 m below the minimum supply level. However, the D_{50} size for the lower part of the riprap can be reduced, because of the shorter fetch at these reservoir levels. This will be studied during final design.

The downstream toe slope of the dam will be subjected to the action of back eddies during spillway and tunnel releases and will require protection. A riprap layer is shown on Plate 24. The riprap protection to be provided on the downstream face of the dam will be based on the results of the model study.

The upstream face of the dyke will be protected with a 1 m thick layer of riprap with a D_{50} size of 500 mm.

8.13 INSTRUMENTATION

The major water retaining structures will be systematically and frequently monitored during and after construction to ensure that:

- the short term performance during construction and immediately after initial filling of the reservoir is acceptable,
- the long term effects of seepage, settlement and wave action are not excessive or detrimental to the integrity of the dam, and
- the need for maintenance or remedial treatment is identified sufficiently early to allow for its orderly and safe execution.

The following instrumentation will be installed in the foundations of the dam, spillway and dyke and within the dam and dyke fill.

8.13.1 Piezometers

Piezometers located in the core and shells will monitor pore pressure buildup and dissipation during construction and reservoir filling. Piezometers in the alluvium, colluvium and rock under the dam, both upstream and downstream of the core trench will monitor pore pressure buildup during construction. Those downstream of the core will also monitor the long term dissipation of the excess over hydrostatic pore pressures. These piezometer results relate directly to the stability of the dam. Piezometers in the bedrock of each abutment will monitor the actual seepage gradients prevailing and will provide data for assessing the need for additional drainage. Pneumatic and standpipe piezometers will be used. Piezometer leads will be taken to central readout stations for the convenience of recording data.

8.13.2 Surface Settlement Monuments

Surface settlement monuments will be installed at regular intervals along the crest of the dam. Survey techniques will be used to monitor these monuments for vertical and horizontal movements. The overall amount, the time rate and the magnitude of differential settlements are of particular interest.

8.13.3 Settlement Gauges

Instrumentation will be installed to obtain information on the distribution of settlement with depth both within the dam itself and within the foundation of the dam and spillway. These data will be used, in conjunction with the surface settlement monument data, to assess the long term settlement effects of the dam.

8.13.4 Horizontal Movement Indicators

Horizontal movements of the dam will be monitored using inclinometers. These additional data will allow the overall deformation of the dam to be assessed.

8.13.5 Seepage Weirs

Seepage surfacing in the abutments will be channelled through weirs to observe and to measure the amount of discharge.

8.13.6 Frequency of Readings

All instrumentation will be monitored regularly in conjunction with an observational program of visual inspection. Observations will be frequent during construction, reservoir filling and initial drawdown and will be eventually less frequent over the years. For a general overview of the performance of the dam and the other structures, the results of the instrument readings will be compared with the predictions based on the design analyses.

SECTION 9

SPILLWAY

9.1 ALTERNATIVE ARRANGEMENTS

In view of the large PMF peak inflow of $9320 \text{ m}^3/\text{s}$ that must be passed through the project, consideration was given to passing the flood flows either by using a single spillway or by using both a service spillway and a fuse plug type of emergency spillway. In both cases the reservoir would be surcharged. The former option was selected. A fuse plug spillway could only be located on the left bank because of the costly rock excavation that would be required on the right bank. The left bank locations shown on Plate 2 are in the region of the buried valley located to the north of the project site. The cost of providing a non-erodible sill for the fuse plug spillway, that would ensure the integrity of the reservoir, was determined to be greater than the cost of enlarging the service spillway. The larger spillway is a better solution as it will provide a more positive control of all flood discharges.

The relative merits of the flip bucket and stilling basin types of energy dissipators were examined. The choice of the flip bucket design was based on its lower overall cost and an assessment that the geological conditions in the discharge area will allow a plunge pool to develop without adverse erosional effects occurring. Implicit in this selection is the necessity for long term monitoring and the possibility of future remedial work.

9.2 SPILLWAY CAPACITY

The 1:1000 year flood of $2,970 \text{ m}^3/\text{s}$ has been adopted as the spillway design flood. The spillway will pass this flood with all gates fully open with the reservoir at the FSL elevation 1,118.6 m.

The results of routing the PMF flood hydrograph through the reservoir is shown on Plate 28. The gates will be operated to maintain the reservoir at FSL. As the inflow to the reservoir increases the gates will be opened more until fully opened. The reservoir will then surcharge above FSL reaching a maximum elevation of 1,124.2 m with a corresponding routed PMF outflow of $7,675 \text{ m}^3/\text{s}$ through the spillway. During the 1:10,000 year flood the reservoir will surcharge to elevation 1119.4 m which is 2.3 m below the reservoir boundary line.

If all inflows in excess of the $80 \text{ m}^3/\text{s}$ capacity of the low level outlet will be discharged through the spillway, it will be used in 9 years out of 10. There is a 70 percent probability that the spillway will be used continuously for a period of two or more months annually.

The adopted arrangement of the spillway is shown on Plates 18, 29 and 30.

9.3 APPROACH CHANNEL

The inlet channel will be excavated mostly through mudrocks. Velocities at the 1:1000 year flood flows have been limited to about 2 m/s. The invert of the channel will be at elevation 1,097 m with a base width of 60 m. Approaching the headworks the channel will gradually widen to 85 m.

The excavated rock slopes will be an average of 0.75 horizontal to 1 vertical, as shown on Plate 30. These will be achieved by using slopes of 0.5 horizontal to 1 vertical with 10 m wide benches at vertical intervals of approximately 20 m. A 10 m wide bench will be provided at the contact of bedrock and overburden, and a 2.5 horizontal to 1 vertical slope will be excavated in the relatively thin soil cover of glacial till and glacio-lacustrine silt and clay. The safety factor of the rock slopes will be close to unity and some instability is expected under drawdown conditions. Although the areas where failure will occur will be localized, substantial volumes of slumped material could be involved. Removal of the detritus could be achieved by dragline from the side but may require lowering of the reservoir. Allowance for such remedial works will need to be made in the operating budgets for the early years of the project during which the slope movement will be expected to occur. In time the slopes will attain a stable configuration. Maintenance of the approach channel to ensure that an adequate hydraulic capacity is available to carry flood flows will be an essential requirement for the safety of the project.

Considerable cost savings in rock excavation and disposal will be realized by adopting this approach. Plate 18 shows the areal extent and magnitude of the excavated requirements for the approach channel and Plate 30 shows the excavated works.

HEADWORKS

As shown on Plates 29 and 30, the headworks will consist of seven 10 m wide bays with the ogee crest 7.5 m below FSL at elevation 1111.1 m. The profile of the nappe is based on a design head of 7.5 m using procedures described in U.S.B.R. "Design of Small Dams". The pointed nose piers will be 2.5 m wide giving an overall width of 85 m.

The headworks will be divided into six weir monoliths and two abutment monoliths by contraction joints located midway between the piers.

Contraction joints will be provided between weir monoliths with PVC waterstops on the upstream face and across the upper face of the crest structure. Shear keys will be formed at the contraction joints to minimize differential movement along the gate structure. Water leaking into the joints will be conveyed by vertical drains into the drainage gallery.

The dimensions and shapes of the piers will be based on hydraulic design, model tests, structural analysis and on equipment requirements.

Reinforcement will be provided as required by the structural analysis and for temperature and shrinkage effects. The piers will be designed laterally as vertical cantilevers, subject to the unbalanced loading resulting from the most severe operating conditions.

The abutment monoliths will be constructed integrally with the ogee section up to the contraction joint at the centre of the end bays. The abutment monoliths will be designed as semi-gravity retaining walls. As in the case of the piers, the abutment monoliths will be designed for the most unbalanced lateral loading that might be experienced.

The approach walls will form part of the transition section between the approach channel and the headworks. The walls will be designed as semi-gravity structures.

9.5 DRAINAGE GALLERY

The drainage gallery will extend under the full length of the gate section. Access to the gallery will be through vertical shafts in each abutment monolith. The shaft on the left side will also provide emergency access to the drainage tunnel under the headworks.

9.6 SPILLWAY BRIDGE

The bridge will have seven spans with a 10 m deck width appropriate for public two-lane traffic. Zero clearance for the PMF surcharged reservoir level will be allowed to the bottom of the deck girders. The bridge deck will consist of a cast-in-place concrete slab supported by precast prestressed girders. The load capacity of the bridge will be sufficient to accommodate public highway requirements, anticipated construction and operating equipment loads and off-highway vehicles.

The 85 m wide chute will generally parallel the natural topography. This will result in a floor slope of 50 horizontal to 1 vertical along the relatively flat uplands and 3 horizontal to 1 vertical down the valley face terminating in the flip bucket. Unit discharges will be about $35 \text{ m}^3/\text{s}/\text{m}$ for the 1:1,000 year flood and $90 \text{ m}^3/\text{s}/\text{m}$ for the PMF. Corresponding maximum velocities will be 30 m/s and 32 m/s respectively. The viability of a convergent chute to reduce excavation and concrete quantities will be studied in the hydraulic model tests. The concrete surface of the chute will be finished to a fine tolerance of irregularities to avoid cavitation damage.

Wall heights will vary from 8.3 m near the ogee to 5.4 m at the beginning of the flip bucket. The chute walls will be of the reinforced concrete cantilever type except adjacent to the headworks where they will be of semi-gravity design. All wall joints will be provided with waterstops and shear keys.

The chute slab will be founded on a 2 m thick layer of free draining gravel. Drainage of the slab from pressure relief holes drilled from the drainage tunnel will be considered.

The excavation for the headworks and chute will also involve large volumes of material as shown on Plates 18 and 30. An average slope of 1.3 horizontal to 1 vertical in excavated rock will be required to provide an adequate safety factor against a deep seated failure. Unlike the approach channel, slope instability above the headworks and chute cannot be tolerated. In bedrock, slopes of 1 horizontal to 1 vertical

excavated with 10 m wide benches at intervals 20 m of height will provide an adequate factor of safety. An excavated slope of 2.5 horizontal to 1 vertical will be used in the relatively thin overburden.

Immediately upon completion of excavation, a 100 mm shotcrete layer or mud slab will be placed on the exposed rock surfaces to inhibit weathering.

The underflow drainage system will consist of the gravel drainage layer containing lateral perforated drainage pipes connected to longitudinal headers at each side of the chute.

Transverse cut-off walls through the gravel drainage layer will be provided at frequent intervals to ensure the drainage flow down the chute is intercepted and directed into the drain headers.

9.8 ENERGY DISSIPATION

Energy will be dissipated by a flip bucket located to discharge spillway flows clear of the downstream toe of the dam. The flip bucket will consist of a 30 m vertical curve terminating at a flip angle of 30°. The radius and flip angle were chosen to minimize hydrodynamic pressures against the chute walls and to reduce the strength of the return currents that will form around the plunge pool. The lip of the bucket will be at elevation 1,062 m which is about 1 m above the estimated tailwater level for the PMF. During the 1:1000 year flood discharge, the impact area will be between 60 m to 80 m downstream from the bucket lip. At lower discharges, it will be closer to the structure and at higher discharges, it will be slightly further away.

The bucket will be anchored to the rock by 150 mm diameter tensioned rock anchors, 50 m long, to accommodate the hydrodynamic and gravity forces acting on it. Pressure relief drain holes will be provided immediately upstream of the flip bucket.

Downstream of the flip bucket minimal excavation will provide drainage towards the river. This will involve some excavation of the valley wall on the right side as shown on Plate 18.

The site investigation results indicate that the bedrock in the vicinity of the plunge pool area is sufficiently competent that pre-excavation of the plunge pool will not be necessary.

The flow characteristics and the erosional aspects of the plunge pool discharge will be studied during the execution of the hydraulic model program to confirm that back eddy currents during any spillway discharge will not damage the riprap protected toe of the dam or undermine the flip bucket structure. The proposed model studies have been advanced to ensure that a solution is available before a final decision is required on tunnel alignment.

The erosional effects of spillway discharges during early years of operation of the spillway will be monitored and should undesirable effects ensue, maintenance measures will be undertaken.

SPILLWAY FOUNDATION

The spillway will be founded on mudrocks containing seams which are located immediately beneath the concrete headworks. These have been assumed to have a residual effective angle of shearing resistance of approximately 16° to which a roughness component of 1.5° has been added. This assumption will be checked by laboratory shear box testing of samples taken from beneath the structure.

The gated headworks structure will resist overturning, sliding and flotation.

The intensities of the uplift pressures at the upstream and downstream extremities of the structure will be equal to the hydrostatic pressures at these locations. The presence of the drainage gallery and the drainage tunnel permits the uplift pressure intensity at the line of the drainage curtain to be taken to be equal to the hydrostatic pressure at the downstream extremity, plus one third of the head differential between the upstream and downstream extremities.

For overturning stability with the reservoir at FSL, the location of the resultant load on the structure base will be within the middle one third of the base width. During PMF conditions, the location of the resultant load will be within the central half of the base width.

For sliding stability during operating conditions, the factor of safety will be greater than 1.5. During PMF conditions, the factor of safety will be greater than 1.2. A check for a "blocked drain" condition, when the uplift pressure is

assumed to vary linearly from the upstream to the downstream extremities indicated that the factor of safety will be greater than unity.

9.10 SPILLWAY GATES

Seven vertical lift roller gates will be provided for the spillway. The selection of this type of gate was based primarily on good history of performance and maintenance, particularly in situations where ice is a major design consideration. Gates will be designed for a 0.6 m ice cover on the reservoir, at any level between FSL and the gate sill. Estimates of gate weights and costs have been based on experience data.

Each spillway bay will be 10 m wide and the gates will be 8.5 m high. For present purposes, three gates have been selected for winter operation. These gates will be insulated and heated by thermostatically controlled blower heaters which force heat through ducting within the gates. The gate guides will be heated electrically. Provision will be made in the guides of the non-heated gates to permit future addition of heaters if required. Detailed assessment of gate heating requirements and possible incorporation of a bubbler system will be included as part of final design.

The gates will be hoisted by wire rope hoist systems located on tower structures on the spillway deck. These will be high enough to permit the gates to be raised and dogged above deck level for maintenance. Housing enclosures will be provided around all mechanical and electrical equipment at the top of the towers.

Local push-button control stations will be provided in locked cabinets at spillway deck level and at the mechanical equipment level at top of the hoist towers. Automatic control by programmed software will be possible using the microcomputer-based control system as described in Section 11. Stop logs will be provided for one spillway bay to allow maintenance of any one gate while the reservoir is at FSL. Stoplogs will be handled by a monorail hoist crane supported by the gate towers.

9.11 INSTRUMENTATION

Instrumentation to record the performance of the spillway foundation is described in Section 8.

SECTION 10
GROUNDWATER CONTROL

10.1 FOUNDATION CURTAIN GROUTING

The primary objective of the curtain grouting will be to seal major open joints in the bedrock and thus minimize water losses from the reservoir and ensure the long term stability of the dam and spillway. Grout will be injected into the rock at intervals between spaced packers until the overall foundation permeability approaches 1×10^{-5} cm/s which is the limit of groutability.

10.1.1 General Layout

The grout curtain design comprises a one to two line curtain 1,000 m long, extending in the river banks from the bedrock surface down to elevation 1,035 m. In the valley bottom the grout curtain will extend from the bedrock surface at around elevation 1,047 m down to elevation 1,000 m. All grouting will be carried out from ground surface except where the schedule allows the dam foundation to be grouted from bedrock surface. Plates 19, 24, and 30 show the grout curtain.

Grouting will also be done from the gallery in the base of the spillway as shown on Plate 29. The grout curtain in the left bank will be tied into the ring grouting carried out from inside the two diversion tunnels as shown on Plate 19.

10.1.2 Length of Grout Curtain

The grout curtain will extend 320 m horizontally into the left abutment measured at elevation 1,052 m. This is a distance of approximately five times the head at the dam, which will theoretically reduce the seepage gradient of reservoir water bypassing the curtain to 0.1.

In the right abutment, the curtain will extend 460 m horizontally, as necessitated by the spillway location.

10.1.3 Depth of Curtain

In the valley bottom, the high permeability zone in the Basal Sandstone Sequence, will necessitate grouting through the entire zone into the Lower Mudrock Sequence to elevation 1,000 m. The mudstones, even with some permeability values measured in excess of 1×10^{-5} cm/s, are expected to provide a good seal underlying the Basal Sandstone Sequence.

The rock in the river banks between elevation 1,040 m and 1,010 m is generally tight and hence the grout curtain need only extend from the bedrock surface through unit K to elevation 1,035 m into the upper part of this zone.

Beneath the valley walls, the grout curtain will slope upward from elevation 1,000 m to 1,035 m at a low angle of about 11° in order to ensure adequate grouting of the potentially pervious stress relieved and jointed rock.

10.1.4 Number of Lines

The high permeabilities in the near surface bedrock, will require a two-line curtain. The lower permeabilities in underlying rock will require only one line of grout holes.

The grout curtain will be positioned to coincide with the third point of the dam core contact zone, measured from the upstream core limit as shown on Plate 24.

10.1.5 Inclination of Grout Holes

To intercept the greatest number of open north-south striking joints, the grout holes in the left abutment will be inclined 30° from the vertical to the east, into the abutment. In the right abutment, the holes will be inclined 30° from the vertical to the west. In the north-south plane, the downstream line of holes will be vertical and the upstream line will be inclined upstream at 10° from the vertical.

10.1.6 Spacing of Grout Holes

Primary, secondary and tertiary grout hole spacings of 6 m, 3 m and 1.5 m respectively are considered necessary to ensure adequate filling of all major open joints. The holes will be drilled full length and grouted from their extremity to the collar by single packer in 3 m intervals.

10.2 TUNNEL RING GROUTING

Under operating conditions, the tunnels upstream of the valve chamber will be full of water at reservoir head, and so ring grouting around both tunnels will be necessary and will form an integral part of the left bank grout curtain. A single

vertical ring of holes positioned in the plane of the main grout curtain will be necessary. The holes will be angled at 15° to each other. The grout holes will follow the contact grouting of the concrete tunnel liner. This is shown on Plate 19.

10.3 FOUNDATION CONSOLIDATION GROUTING

Low pressure consolidation or blanket grouting will be required on a pattern over the entire core contact zone shown on Plate 24. The purpose of consolidation grouting is to seal and strengthen the bedrock directly under the core. Consolidation grouting will comprise holes about 5 m deep on a square grid of 5 m x 5 m spacing.

10.4 PRESSURE RELIEF DRAINS

A pressure relief curtain is considered necessary to ensure stability of the dam and the downstream abutment slopes. Pressure relief is also required at the base of the spillway. to reduce uplift pressures on the headblock. Pressure relief holes will be drilled upwards and downwards from the left and right bank drainage tunnels and downwards from the drainage gallery in the spillway headblock. The location of the drainage tunnels is shown on Plate 18. The tunnel section is shown on Plate 19. The holes will be inclined similar to the grout holes and spaced at 2.5 m centres. The pressure relief holes are shown on Plates 19 and 29.

The drainage tunnels will be lined with a 50 m layer of shotcrete and will have 1.5 m long resin grouted rock bolts on a staggered pattern for final tunnel support. The left bank tunnel will also provide access to the valve chamber. The right bank drainage tunnel will drain the rock in the foundation of the spillway headblock and chute.

SECTION 11
ELECTRICAL AND CONTROL

11.1 ELECTRICAL SYSTEM

A single line diagram showing the electrical system for the project is included as Plate 31.

11.1.1 Power Supply

It has been assumed that two power services will be supplied to the project from the 69 kV high voltage transmission line that currently crosses the project site. This line is currently energized to 25 kV. This will be discussed with TransAlta Utilities Corporation during final design. If possible, a tie breaker connected to both services will be provided to allow load transfer from one service to the other in the event of a power line failure. These services will be supplemented with a 750 kW standby generator set. An automatic transfer switch will be connected to the generator set and to the commercial power switchgear, to provide full back-up power if a total power failure occurs.

11.1.2 Control Building Service

A 100 amp, 347/600 volt, 3 phase subfeeder will be run to the control building to supply heating and air conditioning loads. A 600/120/208 volt, 3 phase transformer will be required to feed lighting loads, receptacles, fire alarm devices, security and control systems.

11.1.3 Spillway Service

A 600 amp, 347/600 volt, 3 phase subfeeder will be run to the gate power distribution center located at the spillway. Gate heaters, gate motors, lighting panels, tunnel and gallery panels distribution center will be fed from this service. The high electrical load required for the gate heating system will be studied during final design to minimize the power demand during the winter as much as possible. The feasibility of reducing the gate heating load by using a bubbler system will be studied.

11.1.4 Low Level Outlet Service

A 200 amp, 120/208 volt, 3 phase power service is required to operate the tunnel valve and gates. Power will feed the valve actuators, tunnel lighting, monorail motor and auxilliary systems. A 15 kW generator set will be installed to provide back up power for control, emergency lighting and the valve actuators.

11.1.5 Tunnel Lighting

The drainage tunnels and drainage gallery will be lit with waterproof anti-corrosion incandescent fixtures. To avoid conduit corrosion, PVC pipes are recommended. Luminaires will be spaced to provide acceptable lighting levels.

11.1.6 Power Lines Relocation

Relocation of the high voltage lines, that pass a close distance from the project site, and cross the river above the tunnel outlet portal, will be discussed and coordinated with TransAlta Utilities Corporation during final design.

11.1.7 Fire Alarm System

A supervised, zoned fire alarm system will be installed in the control building complete with a fire alarm annunciator mounted at the main entrance to allow the firemen to identify the area where the fire has occurred. Storage rooms, return air ducts, stairwells, crawl spaces and kitchen will be protected with suitable devices.

11.1.8 Road Lighting

Galvanized steel tapered type of lighting poles with high pressure sodium luminaires will be used to light the spillway bridge and the access roads leading to the control building and the tunnel access shift. Every luminaire will be furnished with a photoelectric cell to monitor the on/off operations of the luminaires. The poles will be spaced to provide an acceptable lighting level.

11.1.9 Telephone Service

A telephone service for the control building will be designed in consultation with Alberta Government Telephones (AGT). A 50 mm conduit to connect the AGT lines and the main telephone panel inside the control building is required.

11.2 CONTROL AND DISPLAY SYSTEM

Two types of control and display are proposed for control of the spillway gates. The backup or emergency control will consist of a direct wired system, giving the operator the capability of raising or lowering the gates manually while watching analog gate opening indicators. The other will be a

state-of-the-art micro-computer based system which will control the gates and valves, and display pertinent information such as gate and valve position, reservoir level and flow indications as well as all other electrical control and alarm requirements.

The auto control system will consist of dual programmable controllers powered by an uninterruptible power supply. Display will be on a color graphic cathode ray tube. Control will be via keyboard commands.

Communications between the control centre and the tunnel valves will be by dual, separate run, fibre optic cables, installed to eliminate any outside interference such as power induced voltages created by lightning strikes.

Instruments will be used to detect reservoir level, gate and valve position, water flow and any other information pertinent to operation of the dam.

An option compatible with this control system is a printer that could produce alarm and status summaries as well as generate reports on a regular basis.

Remote upstream river level transmitters should be considered for instantaneous level indication for the operator during flood conditions.

11.3 REMOTE METERING STATIONS

Power is needed for the remote upstream and downstream river metering stations to energize the level meters. Level readings will be transmitted to the control building where water flow and discharge flow are requested.

SECTION 12

CONSTRUCTION

12.1 CONTRACT PACKAGING

For the purpose of developing the capital cost estimate and construction schedule, it was assumed that construction of the project would be awarded in seven separate contract packages. These were:

- Package No. 1 - Preliminary Works
- Package No. 2 - River Diversion Works
- Package No. 3 - Dam and Dyke
- Package No. 4 - Spillway Structure
- Package No. 5 - Gates and Valves
- Package No. 6 - Control Building
- Package No. 7 - Site Reclamation

In actuality the Preliminary Works Package will be awarded as a number of smaller contract packages and the gates may be contracted separately from the valves.

12.2 CONSTRUCTION SCHEDULE

Construction of the project has been scheduled over a five year period, commencing in early 1986 and being completed before spring 1991. Reservoir filling would occur during the 1991 run-off season.

The construction period could be reduced one year by diverting the river earlier through one tunnel and allowing spillway excavation and dam construction to start before the second tunnel has been completed. However the contractual implications of delaying the dam contractor, should the second tunnel not be completed in time to pass the following spring flood, are very serious. Consequently, it has been decided to adopt the more prudent approach of starting dam construction after both tunnels have been completed, resulting in a five year construction program.

An outline schedule indicating the major dependencies is shown on Plate 32. For the project to be completed and the reservoir filled during 1991, the Preliminary Works contract package must go to tender by the end of February 1986. This will allow the appropriate temporary facilities and access roads to be usable by the time that the River Diversion Works contractor mobilizes in August 1986. The critical path passes through the diversion works, river diversion, dam foundation excavation, dam fill placement and valve installation. Spillway construction, dyke placement, and spillway gate installation are not on the critical path. Spillway excavation is however linked with construction of the dam. The key phases during river diversion are discussed in Section 6 and are illustrated schematically on Plate 22.

Details of construction sequencing are described in this section together with the work involved with each contract package.

12.3 PACKAGE NO. 1 - PRELIMINARY WORKS

This package provides the construction facilities infrastructure, and includes the main project site access road, major on-site access roads, construction bridges, construction and operation of the construction camp, associated camp utilities, and the engineer's offices and laboratory.

12.3.1 Access Roads

The site access road will be constructed to highway standards with asphalt paving and appropriate entrance arrangements from Highway #3 as shown on Plate 2. A two lane bridge will carry the road across the river.

The site access road passes through the major sand and gravel deposits B530, B540, B550 and B560 shown on Plate 15. These deposits may be developed to supply the processed granular material for filters, drains and concrete aggregates.

On-site access roads will be provided to each tunnel portal and to the spillway headworks area. These are also shown on Plate 2. Consideration will be given to constructing the haul road to the upstream sand and gravel borrow area B50 as part of this contract package. All of the on-site access roads will be designed as all-weather roads with the capacity to carry off-highway vehicles.

12.3.2 Construction Camp

The construction camp will be located on a plateau to the east of the tunnel outlet as shown on Plate 2. The nominal camp capacity will be 500, but provision will be made for an

extension of the camp to 800 should it be required. The kitchen-diner will be sized to handle a daily peak of 800 but will be set up to operate efficiently at a much lower capacity of about 300.

Electricity, propane and water supply and the sewage treatment system will be designed for a capacity of 800.

12.3.3 Miscellaneous

The Preliminary Works contract package will also provide for a catering contractor to operate and maintain the kitchen, diner and camp, the engineer's facilities, the water, power and fuel supply and the sewage treatment works.

12.4 PACKAGE NO. 2 - RIVER DIVERSION WORKS

12.4.1 Cofferdams

The invert elevations of the intake and outlet structures are lower than natural river water levels. Positive cut-offs through the alluvial gravels to bedrock will therefore be required at both tunnel portals to allow complete dewatering of the excavations. A cofferdam with an impervious cut-off trench is proposed for the purposes of the cost estimate. The actual method of sealing the excavations will be left to the contractor and could include a slurry wall cut-off.

12.4.2 Portal Excavation

Initially excavation will be required to prepare the portals for tunnelling. The excavation for the concrete intake and outlet structures will be carried out during the tunnelling

operations. It has been assumed that the open excavation will require drilling and blasting and the estimate is based on the high production rates that it is possible to achieve with hydraulic drills. Spoil from the portal excavations will be used to construct haul roads and cofferdams, or will be wasted.

The overburden at the intake portal will be excavated to the steepest slopes possible that will allow the rock excavation to be safely completed. The balance of the overburden will be removed for dam and dyke fill material as part of the contract package No. 3.

12.4.3 Underground Excavation

The assessment of the geotechnical and geological data currently available permits the following conclusions on underground excavation to be made. Most of these will be checked and verified in the test tunnel described in Section 4.

- Due to the overall rock strength, low stress conditions, comparatively uniform geology and size of openings, major tunnelling problems are not envisaged.
- Excavation could be undertaken using:
 - drill and blast techniques, or
 - a tunnel boring machine (TBM) suited to moderately strong ground.

- The economics in using a TBM will depend on whether a machine of a suitable size is available at the time of construction and has a low mobilization cost. Since the tunnels are relatively short, the high cost to mobilize a TBM may make the use of a TBM non-competitive with drill and blast tunnelling. The requirement to complete excavation of the first tunnel in a three month period may preclude using a single piece of equipment which has the potential for major delays due to equipment breakdown.
- The use and flexibility of a roadheader would offer advantages, however the upper limit of strength of the sandstone component of the bedrock is beyond the capabilities of most available machines.
- For the cost estimate it has been assumed that a full face drill and blast method will be used.

The estimate is based on the following assumed program.

As soon as the inlet and outlet portals have been established in October 1986, excavation of both tunnels will commence and will proceed concurrently. Drilling will be accomplished by the use of a twin boom hydraulic drill jumbo with a basket and a possible third boom for rock bolting. During the mucking cycle the jumbo will be removed. The timing of the operation will be such that the mucking cycle is coordinated with the drill and blast cycle. Mucking from the downstream faces will be carried out using a Haagloader and low profile 25 ton rock trucks.

A second hydraulic drill jumbo will be used at the intake portal for tunnel excavation and again both tunnels will be excavated concurrently. Due to shorter distances to the face, scoop trams could be employed. The assumed equipment has been selected so that should a breakdown occur on either the drill jumbo or the mucking equipment then a corresponding piece of equipment can be moved from portal to portal, until repairs are effected.

12.4.4 Permanent Lining

The estimate and schedule are based on using two, 10.5 m long full circle forms starting near the valve chamber and progressing toward the portals. The east tunnel will be concreted first. Low pressure contact grouting will take place when the lining forms have been removed. Consolidation grouting, will be deferred until the concrete has attained its design strength.

12.4.5 Intake and Outlet Structures

After the intake and outlet excavation has been completed and the tunnel excavation is underway, construction of the concrete intake and outlet structures will start. The transition section of the west downstream portal will be completed at the conclusion of construction of the west tunnel and during diversion of the river through the east tunnel.

12.5 PACKAGE NO. 3 - DAM AND DYKE CONSTRUCTION

12.5.1 First Season (1988)

Package No. 3 consists of dam and dyke construction using materials obtained from the spillway excavation and borrow areas. To meet the requirement for processed granular materials during the of 1988 season, a crushing, screening and washing facility will have to be established in borrow area B50 in the spring 1988. As soon as possible after diversion in July 1988, the downstream drain blanket Zones 2 and 3 will be placed, to allow, the placing of material from the spillway excavation in the downstream Zones 4 and 4A, to commence.

The estimate is based on obtaining the Zone 4 random fill requirements for the 1988 season from the spillway headworks excavation, and completing this structure excavation during the 1988 season. As a substantial portion of the Zone 4 material can be placed prior to river diversion, in the cofferdam and other parts of the upstream shell of the dam, shown on Plate 22, the unsuitable overburden material overlying this source of Zone 4 material must be removed before the spring 1988 flood has subsided and the upstream cofferdam construction can commence. A major haul road will be required from this excavation down the right bank ending at the upstream cofferdam. This haul road will be the main route from the right bank for dam fill materials from the spillway excavation and from the sand and gravel processing setup at borrow area B50. This is shown on Plates 2 and 22.

The glacio-lacustrine silt and clay overlying the spillway headworks area and approach channel will be excavated and hauled with scrapers to the site of a future permanent road as shown on Plate 2. The glacial till exposed by this operation will be hauled to the upstream cofferdam cut-off. Timing may dictate stockpiling of some of this material but this can be combined with moisture conditioning to provide a source of prepared core material by early in 1989.

Rock excavation from the spillway headworks area and glacial till exploitation from the other spillway excavations will proceed to match the fill requirements for the upstream cofferdam and to maximize the volume of Zone 4 that can be placed in the upstream and downstream shells of the dam, as shown on Plate 22. Suitable materials excavated from the dam core contact zone will be placed in Zone 4A of the dam.

Excavation and dam fill placement will cease during the winter months. Grouting of the bedrock through the overburden is planned to commence as early as possible in 1987 and will continue through the winter.

12.5.2 Second Season (1989)

As early as possible in the 1989 season, grouting and foundation preparation of the core contact zone will be completed and placement of the Zone 1 glacial till in the dam core will be initiated. Zone 1 material will be obtained from the spillway excavation to the maximum extent possible consistent with the economics and timing of necessary moisture conditioning. Upon depletion of this source, the left bank borrow area shown on Plate 15 will be used. Glacio-lacustrine silt and clay stripped from the left bank

borrow area with scrapers will be placed in the dyke prior to exposing the glacial till material suitable for Zone 1. It is anticipated that, by this time the dam fill will have reached a level suitable for hauling from the left bank borrow area using the access road to the cofferdam at an elevation above 1085 m. This is shown on Plate 22.

The spillway flip bucket excavation will be completed by May 1989 to allow concrete to be placed to the level of the rock anchor stressing gallery for anchor installation during the winter 1989-1990 under contract package No. 4. The excavated material will be placed in Zone 4 of the dam.

The balance of the Zone 4 material to be placed during the 1989 season will come initially from the spillway chute excavation and finally from the approach channel. Zone 5 pit-run material will be hauled from the B100 borrow area upstream of the dam.

12.5.3 Third Season (1990)

During the 1990 season, the Zone 1 material will be hauled from the left bank borrow area with Zones 4 and 5 material coming from the spillway approach channel and the B100 borrow pit respectively. Processed granular materials will continue to be obtained from borrow area B50. The estimate is based on riprap being quarried from the Blairmore site shown on Plate 16.

12.5.4 Utilization of Excavated Materials

The proposed plan requires almost no double handling of materials. The only exceptions will be approximately 20,000 m³ of glacial till that will have to be stockpiled from the spillway excavation and Zone 1 material that will have to be moisture conditioned. There will be virtually no excavated material that will need to be wasted. The glacio-lacustrine silt and clay on the right bank will be used in the future road gully crossing and the glacio-lacustrine silt and clay overlying the left bank glacial borrow will be used to construct the dyke.

12.5.5 Drainage Tunnels

The drainage tunnels are not on the critical path and have been included in contract package No. 3.

12.6 PACKAGES 4 AND 5 - SPILLWAY STRUCTURE AND GATES AND VALVES

12.6.1 First Season (1988)

It is intended that most of the excavation for the spillway structure will be completed by the dam contractor under contract package No. 3 leaving the spillway chute cut-off trenches, trenches for drain pipes, and the final 300 mm of rock to be completed under contract package No. 4.

Processing of concrete aggregates either from borrow pit B50, or, if the quantity of material proves to be inadequate, from the B530, B540, and B500 sources will be initiated in time to allow concrete placing to commence in April 1989.

12.6.2 Second Season (1989)

During the 1989 season the spillway headworks concrete will have progressed sufficiently to allow the gate contractor to start installing embedded parts in second stage concrete by September 1989. Concrete placing in the flip bucket will be completed to the level of the anchor tensioning gallery to allow anchor installation during the fall and winter of 1989-1990.

The chute underdrain system will be installed during the latter part of the 1989 season.

12.6.3 Third Season (1990)

During 1990 the spillway chute and the balance of the flip bucket concrete will be placed. The gate contractor will install the gates during the summer of 1990. The tunnel plugs will be placed by the spillway contractor and the valves installed starting during the fall of 1990.

12.7 PACKAGE No. 6 - CONTROL BUILDING

The control building will be constructed during the summer of 1989 in readiness for control system installation during 1990.

12.8 PACKAGE No. 7 - SITE RECLAMATION

Site reclamation work will start in the fall of 1990 and will be completed during the spring of 1991.

SECTION 13 COST ESTIMATE

13.1 CAPITAL COST ESTIMATE

The summary of the estimated capital cost of the project is presented in Table 13-1.

13.2 ESTIMATING METHODS

The detailed cost estimate is based on unit prices prepared specifically for the project. The unit price for each major component of the civil works was developed by estimating the labour force, equipment fleet and materials required to complete that particular item of work within an appropriate time frame and then dividing the associated cost by the quantity of work involved. The productivity assumptions made to estimate labour and equipment requirements take into account the project location, current labour management practices and feasible construction procedures. The contractor's indirect costs such as supervisory personnel and site offices were separately estimated and added to allowances for head office overhead, contractor's contingency, profit, insurance, bonds and appropriate financing charges. The total was then distributed to the unit price as a mark-up on direct costs.

For major items of installed permanent equipment, quotations were obtained from manufacturers. These quotations were then used to check costs estimated by means of experience curves developed from projects either built or for which competitive tenders have been obtained.

TABLE 13-1
 OLDMAN RIVER DAM
PRELIMINARY ESTIMATE OF CAPITAL COST
SUMMARY

CONTRACT PACKAGE	DESCRIPTION	ESTIMATED COST (1985 DOLLARS)
1	Preliminary Works	\$ 19,900,000
2	River Diversion Works	\$ 47,800,000
3	Dam and Dyke	\$ 95,900,000
4	Spillway Structure	\$ 54,300,000
5	Gates and Valves	\$ 10,000,000
6	Control Building	\$ 1,300,000
7	Site Reclamation	\$ 1,100,000
TOTAL ESTIMATED CONSTRUCTION COST		\$ 230,300,000
8	Preliminary Engineering and Site Investigation	\$ 6,000,000
9	Design Engineering and Construction Inspection Services	\$ 28,000,000
TOTAL ESTIMATED PROJECT COST		\$ 264,300,000

- Notes:
1. The estimate does not include land acquisition or Owner's administration, insurance, and financing costs.
 2. The estimate does not include reservoir related works.
 3. The accuracy of the estimate is +15%.
 4. The estimate includes 12% allowance for extra work for Contract Packages 1 to 7 inclusive.
 5. The estimate is based on project completion in 1991.

Quantities used in the estimates were taken off the drawings accompanying this report and from the designer's notes and working sketches.

Comparative estimates were made for the alternative layouts shown on Plate 17 by considering only those major quantities that vary from layout to layout and ignoring those items that would be common or that would vary insignificantly.

13.3 BASIC CRITERIA

The cost estimate has been prepared in Canadian dollars using price levels for labour, equipment and materials in effect during the 1985 construction season.

The costs include all appropriate taxes on construction materials and equipment as well as on permanent equipment. The cost of project management, contract administration, and final design was estimated. The cost estimate includes an allowance of 12 percent for "authorized additional work" which, although it cannot be specifically identified at this time, is known from experience to be required to complete projects of this magnitude. It reflects the degree of uncertainty that exists after the completion of preliminary design, and is influenced by the changes that will inevitably result during the detailed final design, the inherent limitations of drilling and other methods used to explore the subsurface conditions and the market climate that could prevail during the period of construction.

13.4 COST DATA

13.4.1 Labour

The following labour rates were used in preparing the estimate.

- Earthworks and tunnel labour rates were based on current Alberta Road Builders agreement rates and conditions.
- Spillway structure labour rates are the union rates currently in effect in northern Alberta. These are based on a \$1.25/h rollback of 1984 agreement rates. These rates are approximately 50 percent higher than currently being paid on small projects in the area. It has been assumed that this project is sufficiently large, that near-union rates will have to be paid to provide an adequately skilled work force for the spillway concrete structure work.
- Gates and Equipment labour rates are the fabrication shop rates currently in effect in eastern Canada.

Separate hourly rate schedules were prepared for each major civil package reflecting the appropriate rates and conditions, a 60 hour work week on a 2 shift basis, and camp catering costs. The fixed costs of camp supply and operation are included as a separate package.

13.4.2 Construction Equipment

An equipment rental rate schedule was prepared for use in developing the direct costs of the project. The rates include all operating and ownership costs. In the case of certain items of plant, such as specialized tunnelling equipment, an adjustment was applied to the indirect costs to

take into account, a lower-than-normal salvage value at the end of the contract, due to anticipated low demand for these items. Operating costs reflect current spare parts, repair, labour and fuel costs. Ownership costs are based on current "transaction" prices obtained from major equipment suppliers and appropriate write-off procedures as recommended by the suppliers.

13.4.3 Materials

Major material requirements were based on quotations provided by suppliers and contractors. The principal material costs are:

Portland cement - Type 50	\$130/t
Reinforcing steel (supply)	\$0.65/kg
Diesel fuel	\$0.35/l

Subcontract budget prices were obtained from suppliers for the supply and installation of grouted rock anchors and the supply of tunnel and cantilever concrete forms.

13.5 VERIFICATION OF CAPITAL COSTS

When the preliminary design was completed, a local general contractor experienced in major civil works and a cost estimating consultant were engaged to prepare two independent check estimates of the major civil works packages Nos. 1, 2, 3, and 4. The results of these estimates confirmed that the capital cost estimate presented in Table 13-1 is reasonable and well within the limits of estimating accuracy.

SECTION 14
POWER BENEFITS

14.1 INTRODUCTION

A preliminary evaluation of the economic benefits of a power installation at the Oldman River Dam was undertaken:

- to enable the minimum provisions for eventual addition of a suitable power installation to be identified and a preliminary cost estimate prepared; and
- to assess whether such a power installation is attractive at the present time.

The evaluation included the following major components:

- an assessment of likely reservoir water levels and releases for three different reservoir operating scenarios;
- a calculation of the energy production for different numbers and sizes of units under different plant operating strategies; and
- an analysis of costs and benefits of the various plant configurations.

14.2 RESERVOIR WATER LEVELS AND RELEASES

14.2.1 Reservoir Inputs

Monthly values of inflows to the reservoir and precipitation on to and evaporation from the reservoir were obtained from Alberta Environment Planning Division for the period 1912 to 1982, inclusive.

14.2.2 Reservoir Releases

Reservoir releases are a function of reservoir inputs, demands and operating policies. The downstream demands were assumed to be comprised of two components:

- riparian flow requirement, and
- irrigation demand.

Riparian Flow Requirement

For the period November 1 through March 31, the irrigation demand is zero but the apportionment agreement and environmental considerations related to fisheries and water quality require some release from the reservoir. For the purposes of this study a minimum flow constraint was adopted based on consideration of the statistics of historical river flows.

Winter flows reported at the WSC gauge (05AA024) near Brocket are summarized in Table 14-1.

TABLE 14-1
WINTER RIVER FLOWS

(m ³ /s)	NOV.	DEC.	JAN.	FEB.	MAR.
Average Monthly	11	9	8	8	13
Minimum Monthly	4	6	4	5	6
Minimum Daily	2.0	2.5	2.8	3.9	4.2

A flow rate of 10 m³/s was nominated as the minimum release for the period November 1 to March 31. This flow rate corresponds to the average flow over the period.

For the period April 1 through October 31, the downstream demands include an irrigation component, and this was supplemented by a minimum of 10 m³/s to satisfy requirements downstream of the LNID headworks.

Irrigation Demand

Monthly values of unit irrigation demand for the LNID area for the period 1912 to 1982 inclusive were obtained from Alberta Environment Planning Division. These values were converted to reservoir release requirements using nominated values of irrigated land area and water delivery efficiency.

14.2.3 Cases Studied

Three cases were examined, each of which gives first priority to supply downstream needs.

Case 1 - Operation of the reservoir in the initial years.

- Releases in the April 1 through October 31 period of 10 m³/s plus irrigation demand rates as used in the South Saskatchewan River Basin Planning Program "Base Case" corresponding to an irrigated district of 70,000 ha in LNID irrigation district were imposed.
- For the period November 1 through March 31 the nominated release of 10 m³/s was maintained.

The simulation of the reservoir was undertaken for the 71 years of available data. It was found that there were no occasions when the reservoir failed to meet its demands and the lowest simulated reservoir level was 1099 m corresponding to about 65 percent depletion of live storage. Also, there were only 2 years (1931 and 1934) when FSL was not achieved during the period spring to early summer.

Case 2 - Operation of the reservoir in the initial years with some portion of storage dedicated to the generation of hydroelectric power.

- Releases in the April 1 through October 31 period would be at the same irrigation demand rates as in Case 1 but November 1 storage would normally be at or above elevation 1107 m.
- Releases in the December 1 through January 31 period would be adjusted to maximize hydroelectric power output but April 1 storage would be assured to be at or above elevation 1095 m to guarantee irrigation and other user obligations during the summer season. This reservoir

level was determined to be the minimum that could occur on April 1 of a given year and still ensure that the reservoir would completely satisfy the summer obligations.

- Releases in November, February and March would be at the nominated minimum rate of $10 \text{ m}^3/\text{s}$.

For this case, in approximately one year out of ten, on average, the reservoir would not be filled in the summer.

The releases in December and January would be constrained by three factors:

- the requirement to have the reservoir at or above elevation 1095 m on April 1 to guarantee irrigation and other user obligations,
- the number and sizes of units, and
- the minimum flow requirement.

Regardless of the configuration of the units, the minimum April 1 water level of 1095 m in the reservoir was maintained, and this provided enough storage to satisfy both irrigation and other user demands and the minimum flow the requirements in all 71 years.

Case 3 - Operation of the reservoir in a mature system.

- Irrigation releases in the April 1 through October 31 period were increased to rates which would result in emptying of the reservoir, on average, once in ten years.
- Nominated minimum release of $10 \text{ m}^3/\text{s}$ in the period November 1 through March 31.

The operation of the reservoir was simulated with various levels of irrigation and other user demands until one was found which did not conform to the above conditions on seven occasions during the 71 years simulated. This is equivalent to meeting the demand ninety percent of the time. This level of demand was found to be equivalent to that an irrigated area of 140,000 ha in the LNID irrigation district. This is twice the area assumed in Case 1. In 12 years out of 71, the reservoir failed to achieve FSL during the spring to early summer period. Accordingly, this simulation was adopted as being a reasonable representation of use of the reservoir in a mature system.

14.3 POWER GENERATION

14.3.1 Hydraulic Characteristics

The hydraulic turbine, that would be installed in a power-plant at the project site, would be required to operate over a wide range of reservoir water levels. A Francis turbine would be most suitable. The principal operating conditions and the corresponding capability of a nominal 15 MW capacity unit are listed in Table 14-2.

TABLE 14-2
15 MW TURBINE OUTPUT

	Net Operating Head	Output Capacity
	(m)	(MW)
Turbine Design Head	54	15
Maximum Operating Head	67.5	21
Operating Head at FSL	66.6	20.6
Minimum Operating Head	34	7.5

Approximately 85 percent of the storage volume of the reservoir will be above the minimum turbine operating level.

At the lowest reservoir levels the flow capacity of the turbines would be greatly reduced. The 15 MW considered in the above illustration would pass $35 \text{ m}^3/\text{s}$ at design head but could not operate with flows less than about $10 \text{ m}^3/\text{s}$ at the minimum head. This study has considered single unit installation of 10 MW, 15 MW and 20 MW and two, three, four and eight unit installations of 15 MW each, in order to maintain the capability of generating power with the $10 \text{ m}^3/\text{s}$ nominated minimum reservoir release.

14.3.2 Limitation on Load Following

In satisfying the required releases of water from the reservoir, the powerplant would produce the greatest benefits by load following or "peaking" operation. This would result in a cycle of water flows which change hour by hour during the day. If carried to excess, peaking operation may prevent

the orderly diversion of water to downstream users and irrigation systems or may cause unacceptable daily flow and water level changes in the river. Accordingly, a policy for plant peaking was adopted which it is felt would keep water level fluctuations less than about 0.7 m in the reach of the river downstream from the powerhouse.

The policy adopted for restricting the amount of plant peaking is defined as an upper limit to the amount of water released in the assumed two-hour peaking period and this was set at four times the average flow release for the day. For example, if the average for the day was $10 \text{ m}^3/\text{s}$, the release during the peaking period would be, at most, $40 \text{ m}^3/\text{s}$ and would be about $7 \text{ m}^3/\text{s}$ during the off-peak period. The actual release would also be constrained by the size of the units and the head in the reservoir.

14.3.3 Alternative Peaking Procedures

The potential power generation of the hydro installation was estimated for the three alternatives of unrestricted peaking, no peaking and restricted peaking as determined in Section 14.3.2. The restricted peaking case represents a reasonable approximation to the most realistic operating mode of the powerplant, and the other two cases, unrestricted peaking and no peaking, were included to establish the effects of these operating modes on the economics of a powerplant installation.

14.3.4 Representative Values of Estimated Production

Capacity and energy benefits estimated for the case of dedicated storage and restricted peaking are plotted on Plate 33 for total plant installations ranging from 10 to 120 MW.

14.4 ECONOMIC ANALYSIS

14.4.1 Benefits

The power benefits for the project comprise three components:

- capacity value;
- energy value; and
- synchronous condenser value.

The Electric Utility Planning Council (EUPC) has considered the possible role of an Oldman River powerplant in meeting electricity needs in the province and has nominated the following preliminary values for evaluating benefits:

Capacity Benefits

Dispatchable and available over the peak hours:

December 1 to January 31 - \$6.10/KW/month/year
February 1 to November 30 - \$1.60/KW/month/year.

Energy Benefits

December 1 to January 31 on-peak - \$20/MWh,
December 1 to January 31 off-peak and
February 1 to November 30 - \$10/MWh
(blended price.)

Escalation

Real escalation after 1991 at 1 percent per year.

Synchorous Condenser

A value of \$200,000 per 15 MW has been allowed for displacing static capacitor compensators which otherwise would be installed. This is partly offset by the value of operating losses taken at 20 kW per MW during the period of time when the plant is not generating power.

Transmission

It has been assumed that energy savings which would result from a generating station located in the Lethbridge area, would reduce the electrical losses in the existing transmission lines to the area by 4 percent of the energy generated. This amount has been added to the value of the energy benefits.

The annual value of benefits estimated for plant installation of from 10 MW to 120 MW are plotted on Plate 33.

14.4.2 Estimated Capital Costs

Powerplant Installation

The powerhouse would be located immediately downstream of the tunnel outlet portal as shown on Plate 34. For the purposes of illustrating the manner in which a powerplant would be included in the project a two unit installation with 15 MW

generators coupled to a 23,000 Hp Francis turbine with a rated head of 54 m is shown on Plate 34. Other installations would be similar as can be seen from the table of major powerhouse dimensions on Plate 34.

The machine bay would be of conventional design and layout with upstream piping and drainage galleries, and a downstream cable gallery. The generator floor area would be serviced by a travelling bridge crane.

The low voltage switchgear would be housed on the upstream side of the powerhouse immediately below the main transformers. The high voltage switchgear and the take-off tower would also be located on the upstream side of the powerhouse. It has been assumed that the outgoing line would be connected to the existing 69 KV transmission line crossing the project site.

The repair bay is sized to provide adequate space for machine dismantling during emergency repair and routine overhaul works.

Flow would be conveyed to the unit by a 4.5 m dia penstock which would be installed in the tunnel downstream of the plug. Emergency closure of the flow would be provided by a gate valve immediately downstream of the tunnel plug. Routine flow closure would be effected by operation of the powerhouse valve.

Construction

Installation of the powerplant would require the construction of a reinforced concrete wall that would exclude the irrigation and riparian flows from the powerhouse area. A cofferdam would also be placed downstream of the powerhouse to seal the area from the river. The cofferdam may be of timber crib construction due to space considerations.

Capital Cost Estimate

Table 14-3 shows the estimate of capital cost for the different powerplant installations considered during the study.

TABLE 14-3
SUMMARY OF ESTIMATED CAPITAL COSTS

		Estimated Cost (\$ x 10 ⁶)			
Installed Capacity (MW)	1-10	1-15	1-20	2-15	
Civil Works	3.4	4.3	5.4	7.1	
Mechanical and Electrical Equipment	5.6	7.8	8.9	11.3	
SUBTOTAL	9.0	12.1	14.3	18.4	
Engineer and Owner Costs	0.9	1.2	1.4	1.8	
SUBTOTAL	9.9	13.3	15.7	20.2	
Contingency (8%)	0.8	1.1	1.3	1.6	
TOTAL	10.7	14.4	17.0	21.8	

Estimated Operating Costs

Operating costs have been estimated on the basis of \$1.75 per MWh of energy generated plus 20 kW per MW during the period of time when the plant is not generating power (synchronous condenser operation). The operating costs were obtained from the U.S. Department of Energy "Hydroelectric Power Evaluation" 1979.

14.4.3 Economic Analyses

Economic analyses have been carried out to guide the choice of an optimum plant capacity. A set of analyses for different plant capacities was based on the following conditions:

- restricted peaking operation;
- dedication of 135,000 dam³ of live storage to winter power operations during the first 15 years of plant operation
- decline of annual benefits from the "initial years" to the "mature system" conditions over the period of the 16th to the 25th year; and
- rate of real inflation of energy costs of 1 percent per year after 1991.

The results of the analyses for various sizes of power installations are shown on Plate 33 in terms of three parameters:

Present Value of Net Benefits has been computed as the sum of the present value of the net benefits estimated in each year from 1986 through 2040, discounted to 1985 at 6 percent per year.

Benefit to Cost Ratio has been computed as the ratio of the present value of benefits to the present value of costs, both discounted at 6 percent per year.

Internal Rate of Return has been computed as the discount rate at which the present value of the stream of costs and the stream of benefits are equal.

The 30 MW installation (two 15 MW units) yields the greatest value of net benefits. However, the benefit to cost ratio and internal financial rate of return indicate that an installation of about 10 MW is optimum. It is felt that an installation of less than 10 MW would be impractical and thus the optimum plant capacity is in the range 10 MW to 30 MW. For the purposes of this study the two 15 MW unit installation has been adopted as optimum since it provides the maximum net benefits.

The features of the nominated installation are summarized in Table 14-4.

TABLE 14-4
FEATURES OF THE NOMINATED INSTALLATION

Number of Units	2
Nominal capacity (at 54 m design head)	30 MW
Maximum Output (at 67.5 m head)	42 MW
Minimum Output (at 34 m head)	7.5 MW
Estimated Investment Cost	$\$21.8 \times 10^6$
Estimated Annual Operating Cost	$\$0.27 \times 10^6$
Average Annual Energy Production	
1991 - 2005	145 GWh/yr
After 2015	135 GWh/yr
Firm Capacity Provided	
1991 - 2005	25.0 MW Dec. - Jan.
	25.0 MW Feb. - Nov.
After 2015	9.2 MW Dec. - Jan.
	19.8 MW Feb. - Nov.
Power Factor Correction Provided	30 MVAR
Annual Value of Benefits	$\$ 2.25 \times 10^6$ per year
Present Value of Net Benefits	
Discounted at 6 percent	$\$ 7.2 \times 10^6$
Benefits-to-Cost Ratio at	
6 percent discount rate	1.4
Internal Rate of Return	9.3%
Percent of potential energy of the	
site which would be developed	approximately 60 percent.

14.4.4 Sensitivity Analyses

The benefits and economic performance of the nominated installation have been tested for various changes in the basic assumptions. A comparison of the sensitivity analyses results is presented in Table 14-5.

TABLE 14-5
SENSITIVITY ANALYSIS FOR THE NOMINATED INSTALLATION

	PRESENT VALUE OF NET BENEFITS (1)	TOTAL BENEFIT TO COST RATIO (2)	INTERNAL RATE OF RETURN
	(\$1986 x 10 ⁶)		(%)
Nominated installation	7.2	1.4	9.3
Without dedicated storage	7.1	1.4	9.3
With no peaking	3.5	1.2	7.6
With unrestricted peaking	8.3	1.4	9.8
Irrigation demands reach mature levels by year 2015	6.4	1.3	9.0
Plant installed when mature system prevails	4.1	1.2	7.8
Zero percent escalation of energy benefits	4.8	1.2	8.4
Two percent escalation of energy benefits	9.6	1.5	10.1

NOTES: 1. Based on 1 percent escalation.
2. Discounted at 6 percent.

Value of Dedicated Storage

The analysis was repeated for the two x 15 MW installation assuming water releases from the reservoir would be controlled solely by downstream water needs. The results indicated that the present value of net benefits was reduced by approximately 1 percent.

Value of Restricted Peaking

If it is assumed that there could be no peaking operation of the powerplant, the net benefit would be reduced by 51 percent.

Restrictions on Peaking

It was found that the net benefits would increase by 15 percent if there were no restrictions on peaking.

Timing of the Irrigation Demands

The analysis was repeated assuming dedicated storage for 10 years instead of 15 and that the reservoir would thereafter be fully utilized by 2005. This showed that the net benefits would be reduced by 11 percent.

Timing of the Powerplant Installation

To determine the sensitivity of the timing of the powerplant installation, the analysis was repeated using only long term benefits from 1991 to 2040. The results showed that the net benefits would be reduced by 43 percent.

Energy Value Escalation

At zero escalation the net benefit would be reduced by 33 percent and at 2 percent escalation the net benefit would be increased by 33 percent.

14.5 WIND OPERATED PUMPS

The possible benefits of pumping water from the tailrace to the reservoir using wind operated pumps was evaluated. The pumping capability of a wind pump is a function of both the size of the installation and the available wind energy. Wind power is a function of velocity cubed and a weighted mean velocity = $(1/n (V_1^3 + V_2^3 + \dots + V_n^3))^{1/3}$ was estimated as 24 km/hr for the Pincher Creek area using the Atmospheric Environment Services wind records for the period 1973-1980. Information provided by Wind Power Inc., data for their model 65 V windpump was extrapolated from the model 50 windpump data by assuming power production is proportional to the swept area.

A model 65 V windpump could pump approximately 3.4×10^6 m³/yr of water from the tailrace area back into the reservoir. Under the most favourable conditions, this water could produce about 0.53 GWh per year. The average energy value for the optimum plant size of two 15 MW is 15 mills and applying this value leads to the total annual benefit of some \$8,000. Allowing for a real discount rate of six percent leads to a present worth of \$97,000 over a project life of 50 years. The cost of installing a model 65 V windpump at the site is \$450,000 and the discounted cost from 1991 to 1985 is \$315,000. Ignoring maintenance costs and any

alterations to the hydroelectric powerplant facilities that may be required, the benefit to cost ratio is about 0.3. It appears therefore, that water pumping through wind power is not economically viable for this site.

14.6 CONCLUSIONS

It has been determined that the optimum plant size is in the range 10 MW to 30 MW. A financial rate of return on investment of approximately 9.3 to 11.5 percent has been estimated. The following supplementary conclusions can be drawn from the analysis.

- Dedication of part of the reservoir storage to power operations in the early years of the project does not lead to a worthwhile increase in benefits.
- The value of benefits is very sensitive to the future real rate of escalation of energy costs. A differential escalation rate of 1 percent was used in the analysis. Net benefits would be approximately 33 percent less if no escalation were assumed.
- The ability to operate the plant on a moderate peaking cycle accounts for approximately 50 percent of the benefits.

The economic analyses indicate that the installation of a small hydroelectric generating station is financially attractive and is worthy of further investigation.

14.7 MINIMUM POWERPLANT PROVISIONS

If a decision is made to provide facilities in the project to allow for the future installation of a powerplant, the following minimum measures would need to be included during the initial project construction. These are illustrated on Plate 34 for the two 15 MW installation.

- Embedment of a steel liner in the tunnel plug of the same diameter as the steel penstock that will be installed in the future.
- Provision of a flange or other means of connecting the end of the steel liner to the future penstock.
- Provision of a larger guard gate to operate in the larger diameter pipe through the tunnel plug.
- Inclusion of a steel transition from the larger diameter tunnel plug pipe and guard gate to the 2 m diameter hollow cone valve required for irrigation and other summer releases.
- Enlargement of the valve chamber and access shaft to accommodate the increased length of the guard gate-transition-cone valve system and to allow for the installation, servicing and removal of the larger guard gate.
- Addition of a reinforced concrete wall between the two tunnel outlets that will form part of the powerhouse excavation cofferdam.

The estimated cost of these additional items in 1985 dollars is \$1,250,000.

BIBLIOGRAPHY

This is not a complete list of references used. The references listed are those that relate specifically to the project and those that were used for the preliminary design but are not listed in the Appendices.

Alberta Environment, (1984), "South Saskatchewan River Basin Planning Program Summary Report".

--- (1983), "Dam and Canal Safety Guidelines." Publication D-S 3.

--- (1983), "Water Balance and Simulation Model User Guide".

--- (1982), "South Saskatchewan River Basin: Historical Natural Flows, 1912 to 1978".

--- (1976), "Oldman River Flow Regulation, Preliminary Planning Studies", (Volumes I to V).

--- (1975), 1975 "Flood, Oldman and South Saskatchewan River Basin".

Basham, P.W., Weichert, D.H., Anglin F.M. and Berry M.J. (1982), "New Probabilistic Strong Seismic Ground Motion Maps of Canada. A Compilation of Earthquake Source Zones, Methods and Results". Earth Physics Branch Open File 82-33.

Bishop, A. W. (1955), "The Use of the Slip Circle in the Stability Analysis of Slopes". Geotechnique, Vol. 5.

Bishop, A. W. and Morgenstern, N. R. (1960) "Stability Coefficient, for Earth Slopes", Geotechnique, Vol. 10.

Environment Canada (1985), "Sediment Station Analysis Oldman River Near Brocket 05AA024", Inland Waters Directorate Western and Northern Region.

Fread, D.L., (1979), "The NWS-Dam-Break Flood Forecasting Model", National Weather Services, Maryland.

Hoek, E and Brown, E.T. (1980), "Underground Excavation in Rock" Institute of Mining and Metallurgy.

Janbu, N. (1956), "Stability Calculations for Fills, Cuts and Natural Slopes". Norwegian Geotechnical Institute, Publication 16.

Janbu, N. (1973), "Slope Stability Computations in Embankment Dam Engineering", (Hirschfeld, R.C. and Poulos, S. J. eds.) Wiley, New York, 1973, pp. 47-86.

Jerzykiewiz, T. and Sweet A.R. (1985), "Preliminary Report on Stratigraphical Setting of Oldman River Dam", Geological Survey of Canada.

McKay, G.A., (1967), "Meteorological Conditions Leading to the Project Design and Probable Maximum Flood on the Paddle River", Canada Dept. of Transport, Meteorological Branch, Toronto.

Mollard, J.D., (1985), "Preliminary Report on the Geology of the Oldman Dam Reservoir Area".

Morgenstern, N. R. and Price, V.E. (1967), "A Numerical Method for Solving the Equations of Stability of General Slip Surfaces", Computer Journal, Vol. 9, No. 4., pp. 388-393.

Morgenstern, N. R. and Price, V.E. (1965), "The Analysis of the Stability of General Slip Surfaces", Geotechnique, Vol. 15, No. 1, pp. 79-93.

Morgenstern, N. R. (1963), "Stability Charts for Earth Slopes during Rapid Drawdown" Geotechnique, vol. 13.

National Academy Press, (1983), "Safety of Existing Dams, Evaluation and Improvement".

National Academy Press, (1985), "Safety of Dams, Flood and Earthquake Criteria".

Oldman River Basin (1978), "Phase Two Studies: Hydroelectric Energy Potential", Energy Resources Conservation Board.

Oldman River Basin (1978), "Phase Two: Water Supply Alternatives", Kohn Leonoff Consultants Ltd.

Phinney, R.B. (1982), "Flood of June 1975 in the Oldman River Basin, Alberta". Environment Canada.

Prairie Farm Rehabilitation, (1974), "Oldman River Basin, Water Supply Study", Hydrology Report 80.

--- (1963), "Preliminary Geological Report on the Three Rivers Damsite, Oldman River, Alberta".

--- (1965a), "Preliminary Soil Mechanics Report, Oldman River Project, Three Rivers Damsite".

--- (1967b), "Bedrock Outcrops in Immediate Vicinity of Castle River Railway Crossing". Plan 17845.

--- (1967c), "Bedrock Outcrops in Immediate Vicinity of Spillway, Three Rivers Damsite". Plan 17911.

--- (1967d), "Surface Geology in Vicinity of Damsite, Three Rivers Damsite". Plan 17977.

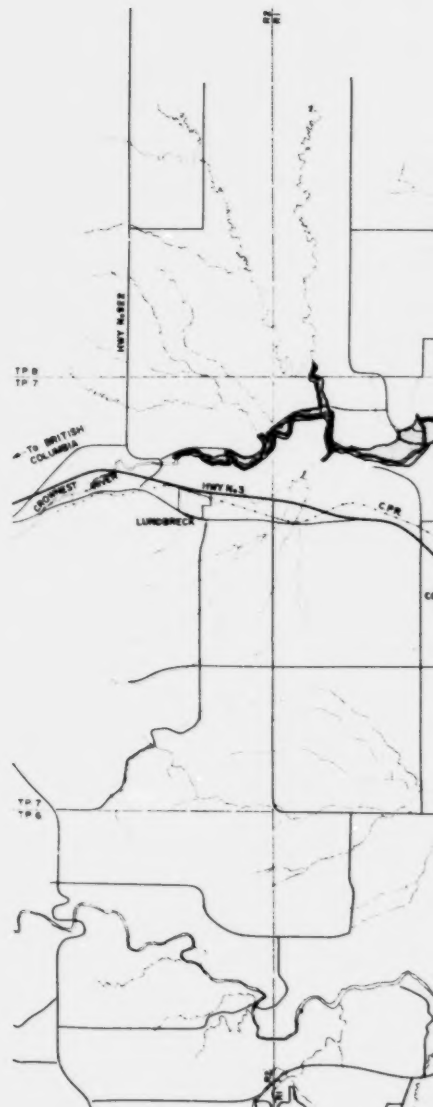
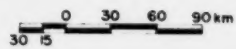
- (1974), "Inventory of Existing and Proposed Projects in the Oldman River Basin." Hydrology Report #76.
- (1964), "Statistical Estimates of Probable Maximum Rain Fall in the Prairie Provinces", Revised Jan.
- Stolte, W.J. and Dumontier, G.R., (1977), "Flood Frequency Analysis for Mountainous and Prairie Streams", Research Contract Report for Alberta Environment.
- Storr, D., (1963), "Maximum One-Day Rain Fall Frequencies in Alberta", Meteorological Branch - Dept. of Transport, Canada.
- (1967), "A Frequency Analysis of Maximum Two-Day and Three-Day Rain Falls in Saskatchewan, Alberta, and Northeastern B.C."
- U.S. Army Corps of Engineers, (1982), HEC1. "Flood Hydrography Package", Users Manual.
- (1982), "HEC 2. Water Surface Profiles", Users Manual.
- "HEC 6. Scour and Deposition in Rivers and Reservoirs", Users Manual.
- (1953), "The Unified Soil Classification System". Technical Memorandum 3-357.
- U.S. Bureau of Reclamation, (1981), "Criteria for Selecting and Accommodating Inflow Design Floods for Storage Dams and Guidelines for Applying Criteria to Existing Storage Dams", Technical Memorandum No. 1.
- (1984), "Probable Maximum Precipitation Estimates - United States Between the Continental Divide and the 103 Meridian". Hydrometeorological Report No. 55,
- U.S. Department of the Interior, (1977), Bureau of Reclamation, "Design of Small Dams", Water Resources Technical Publication.
- Verschuren, J.P. and Wojtiw, L., (1980), "Estimate of the Maximum Probable Precipitation for Alberta River Basins". University of Alberta.
- Water Resources Branch, (1963), "Flood of June 1953 in the South Saskatchewan River Basin". Water Resources Paper No. 113F, Water Management Service, Environment Canada.
- Warner, L.A., (1973), "Flood of June 1964 in the Oldman and Milk River Basins, Alberta", Environment Canada, Tech. Bulletin No. 73 Ottawa.

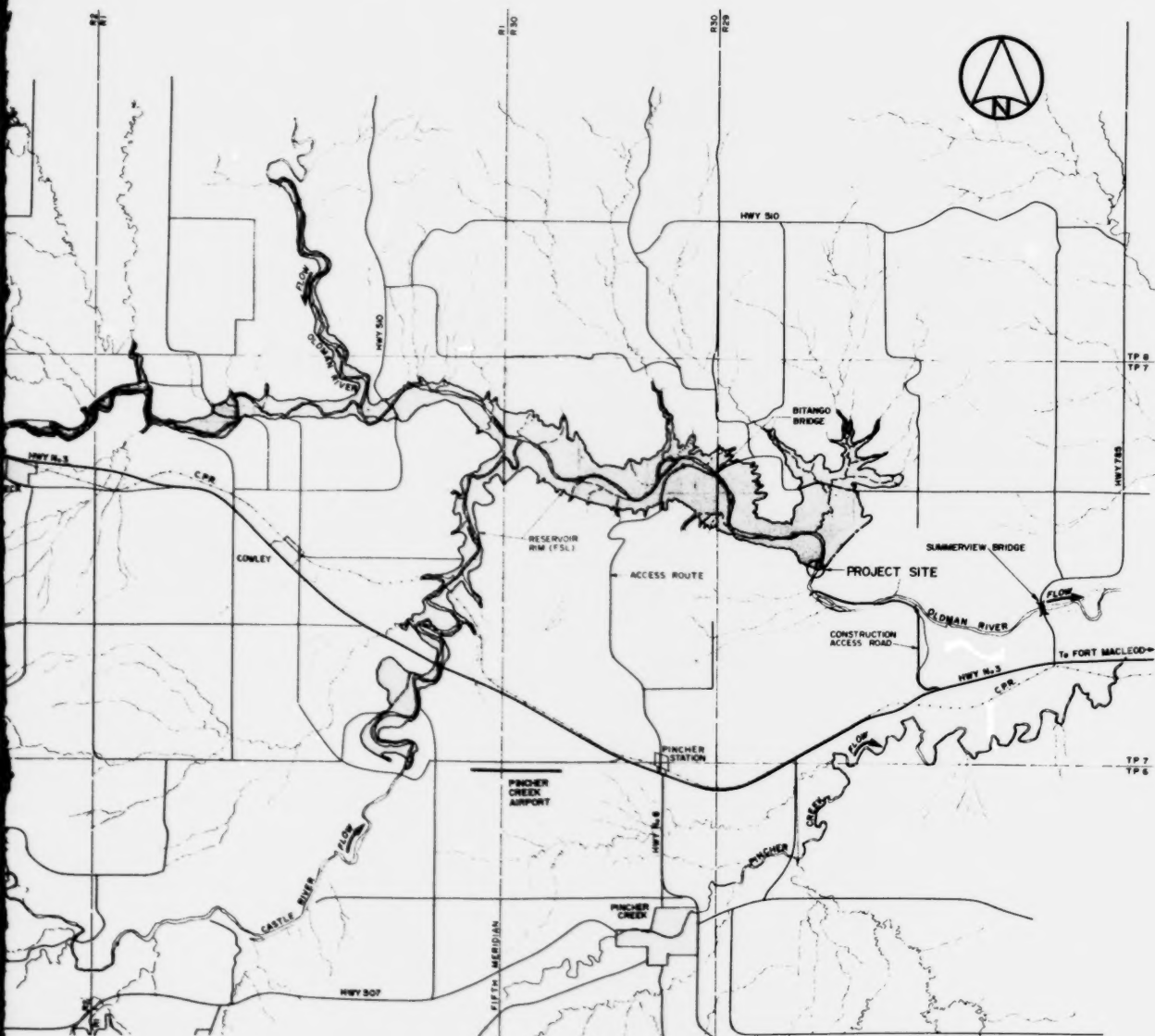
World Meteorological Organization, (1973), "Manual for Estimation of Probable Maximum Precipitation", United Nations Operations Hydrology Report No. 1.

--- (1973), "Water Resources Series No. 30, Assessment of the Magnitude and Frequency of Flood Flows". United Nations.



LOCATION MAP





PROJECT AREA



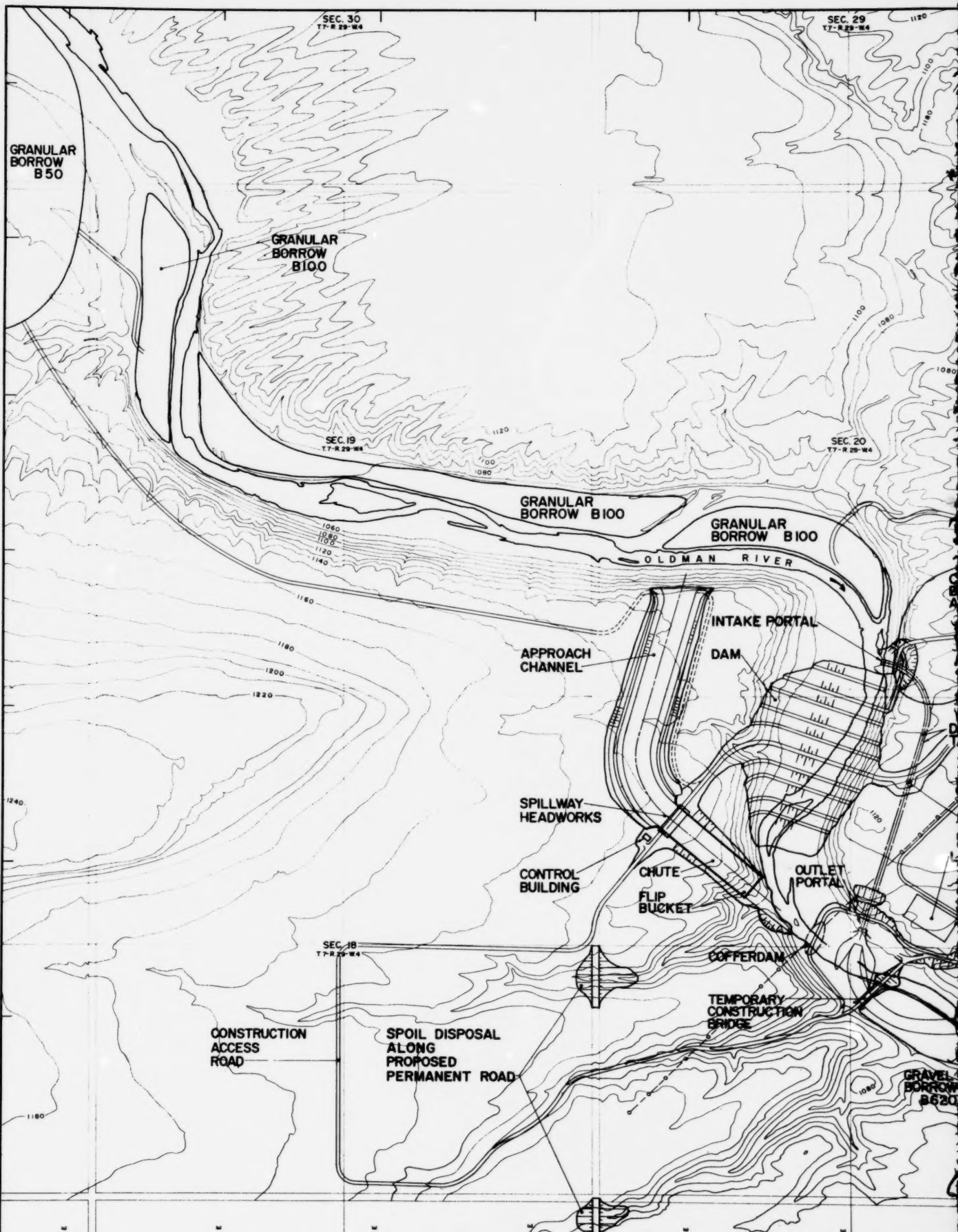
Alberta
ENVIRONMENT

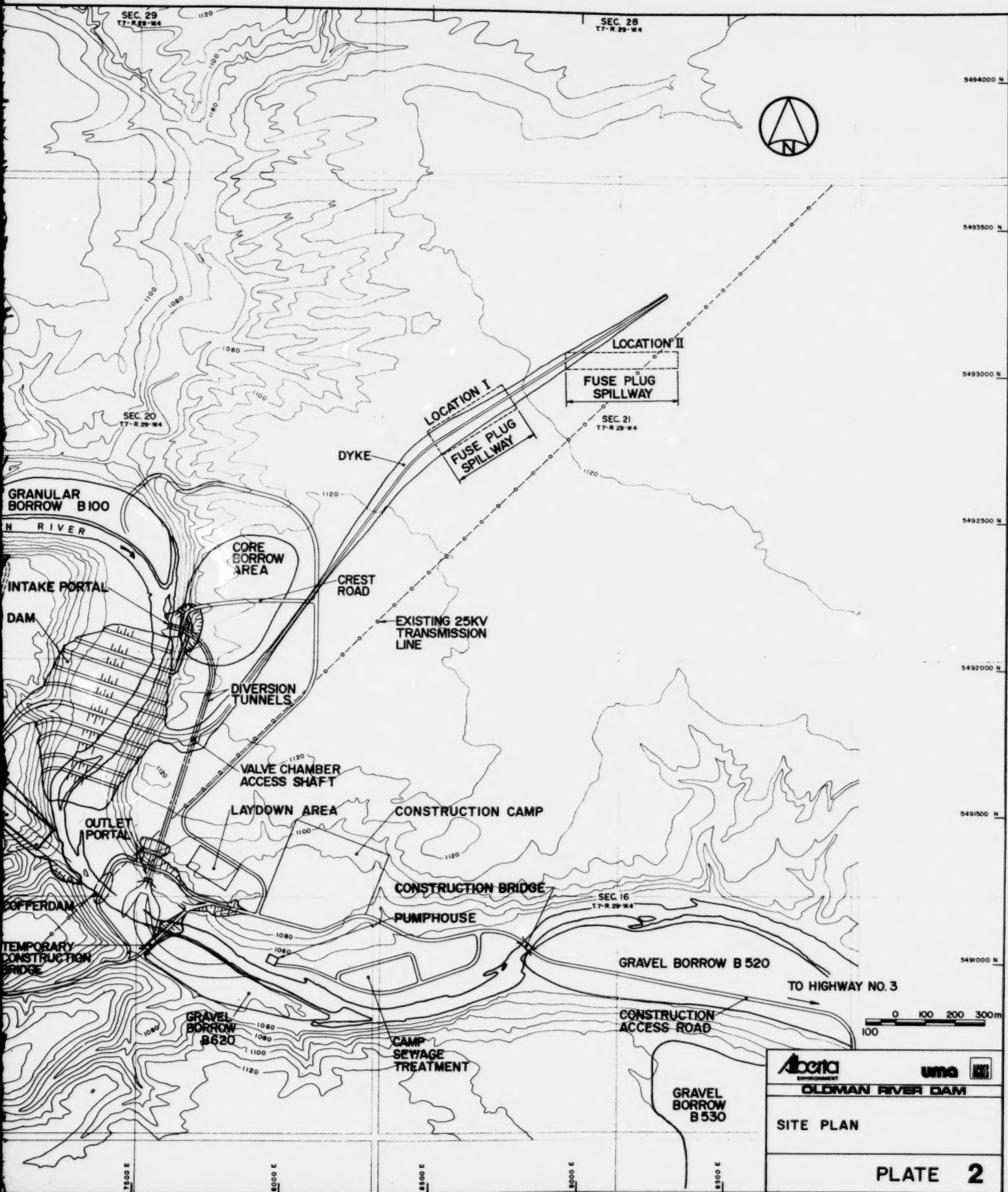
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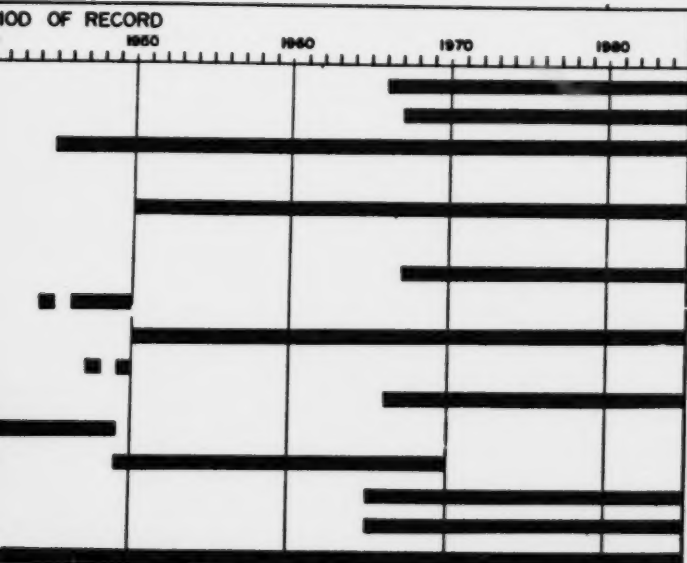
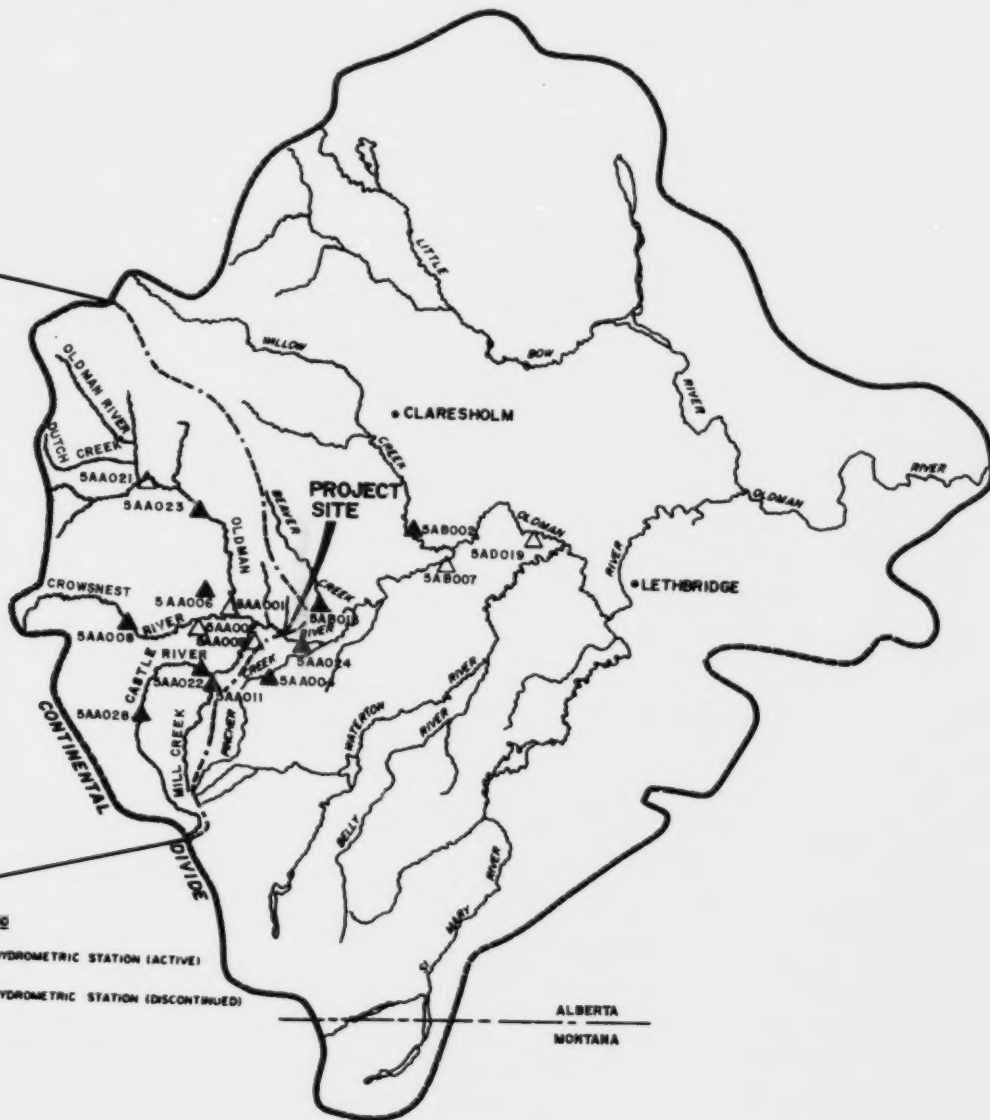
OLDMAN RIVER DAM

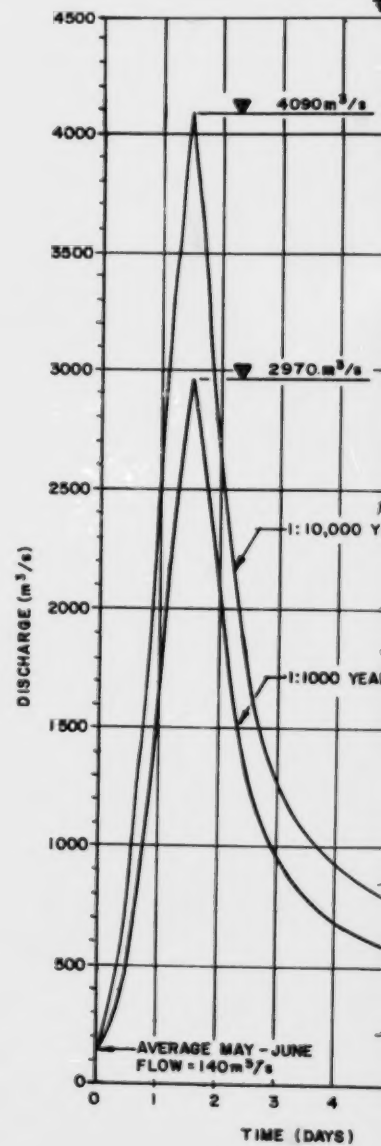
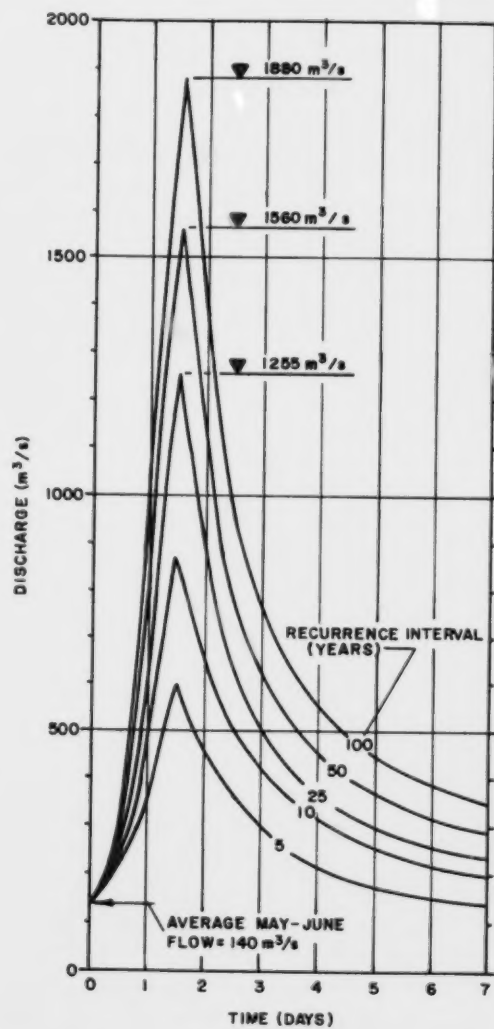
PROJECT LOCATION

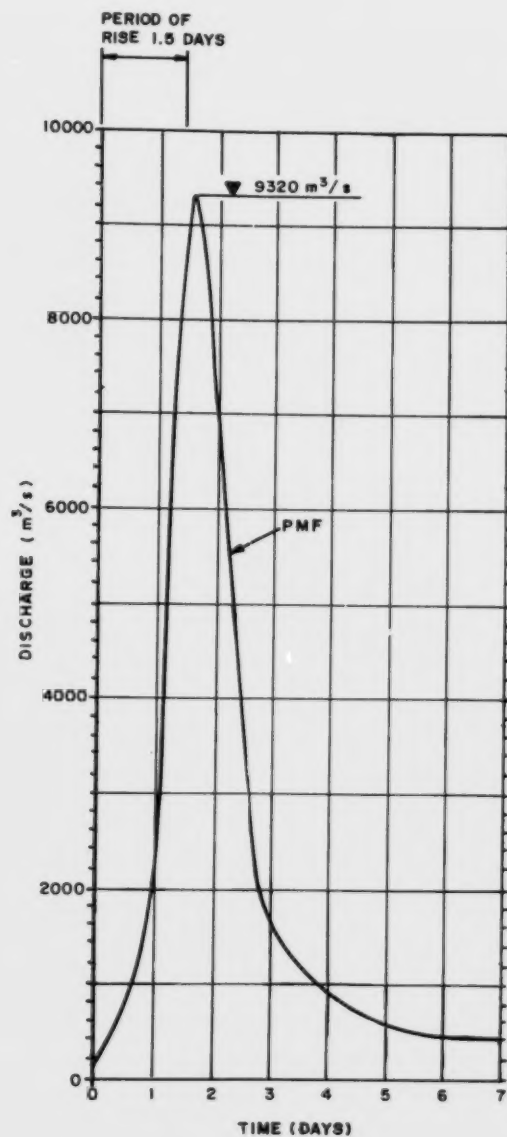
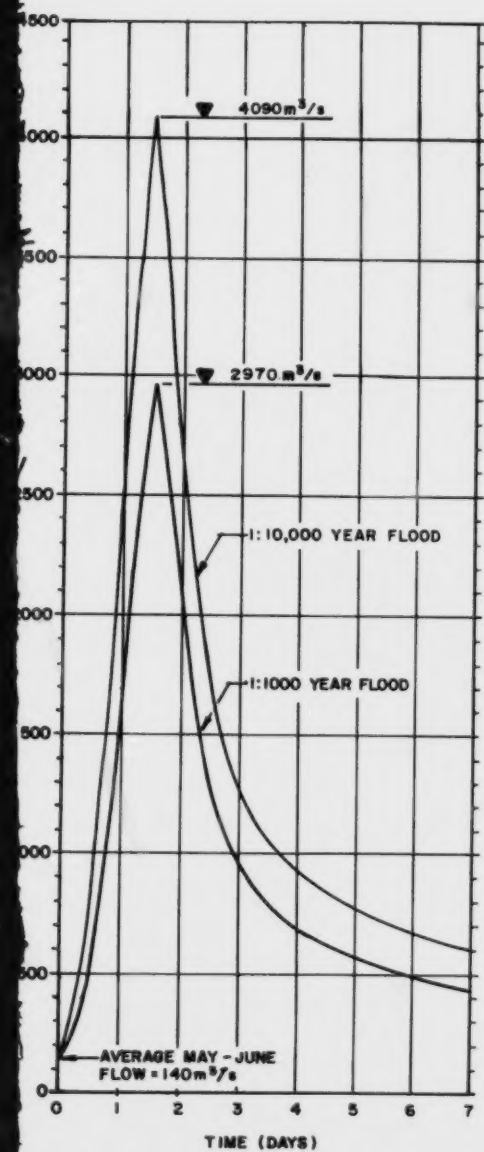
PLATE 1

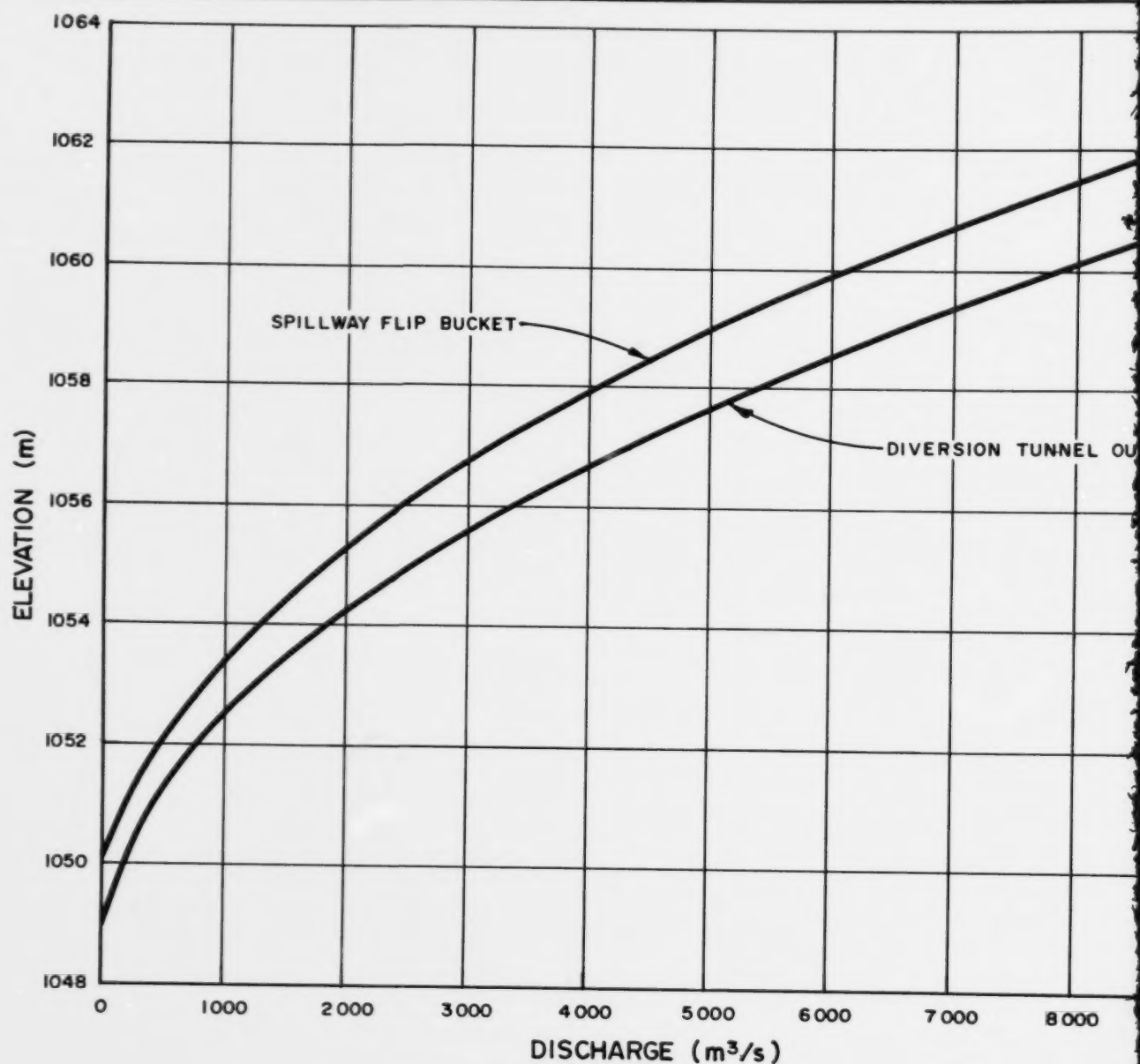




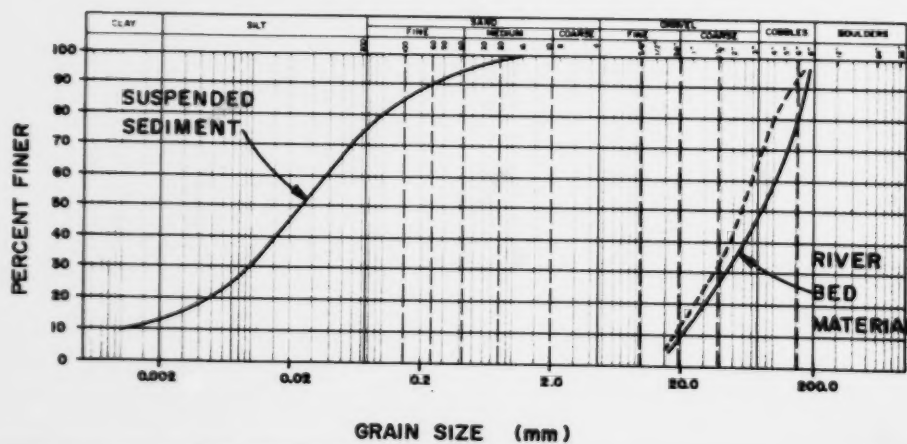


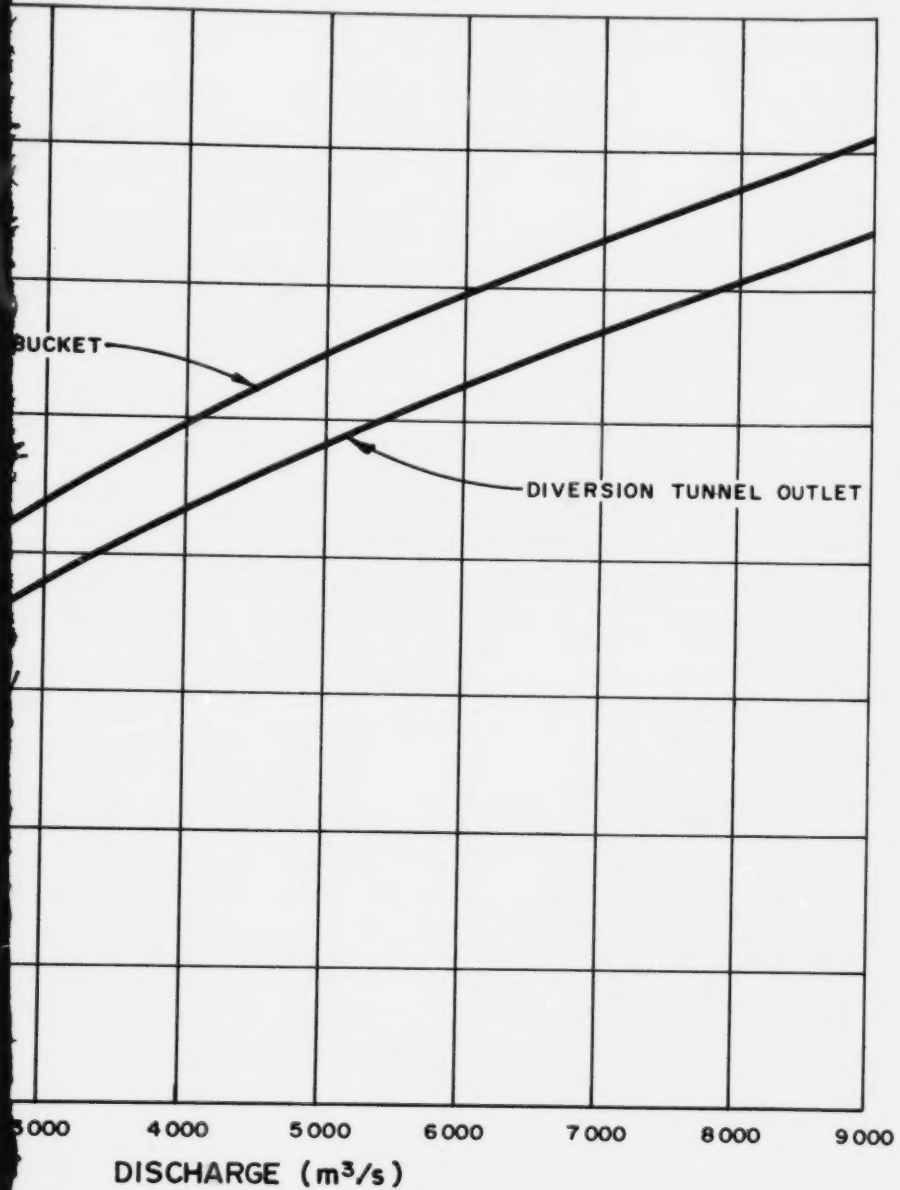




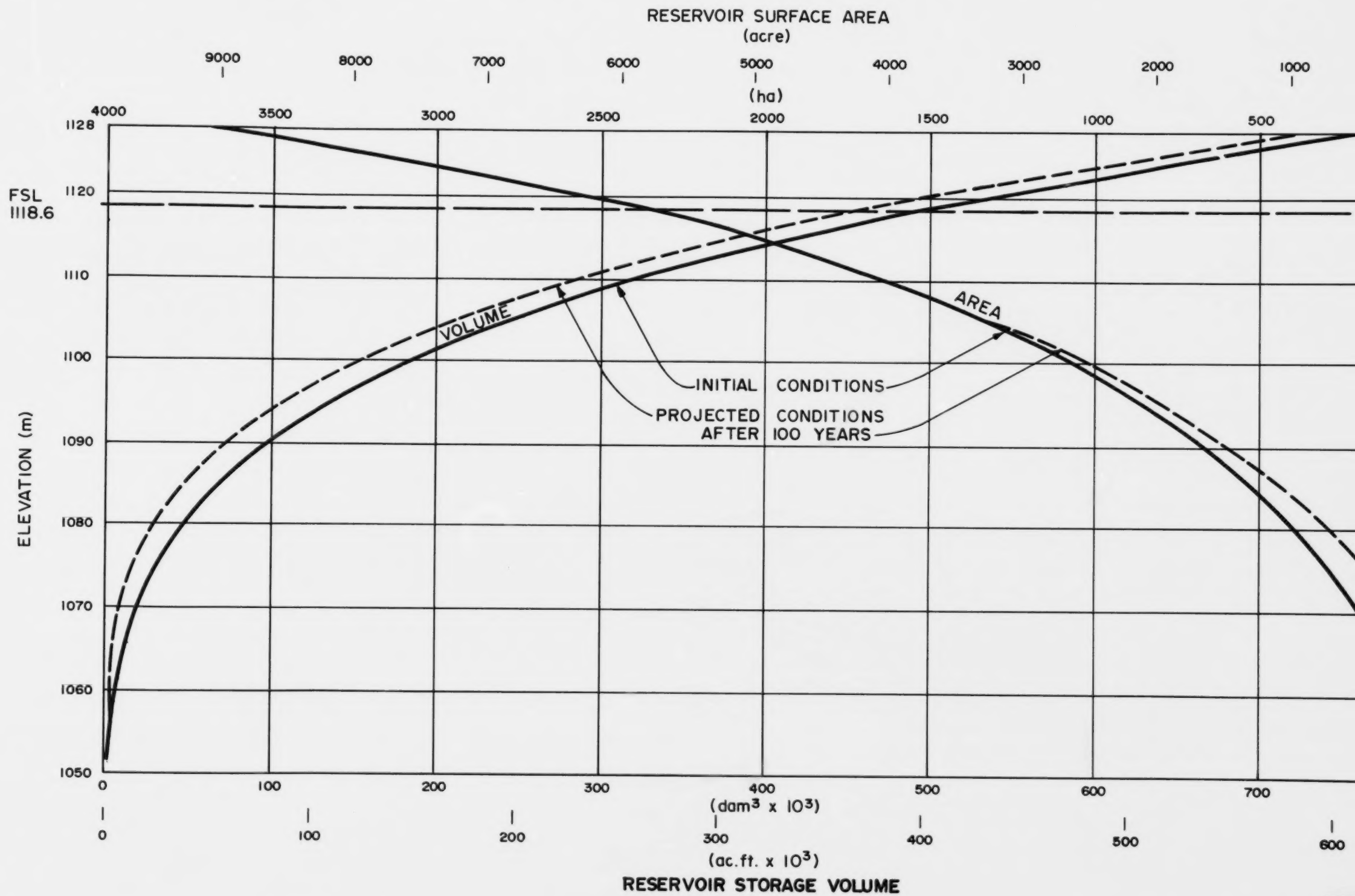
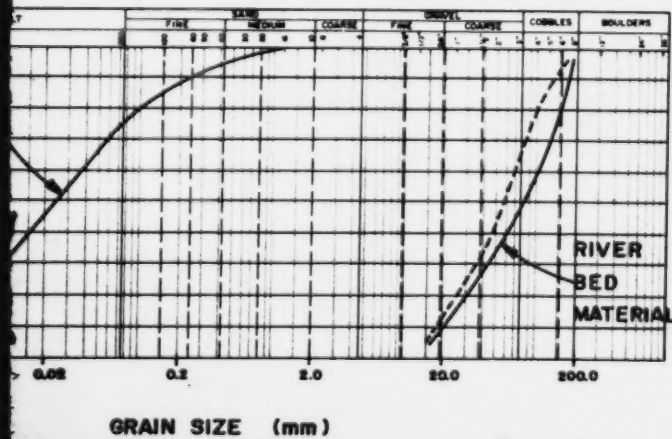


TAILWATER STAGE DISCHARGE CURVES





WATER STAGE DISCHARGE CURVES



RESERVOIR SURFACE AREA (acre)

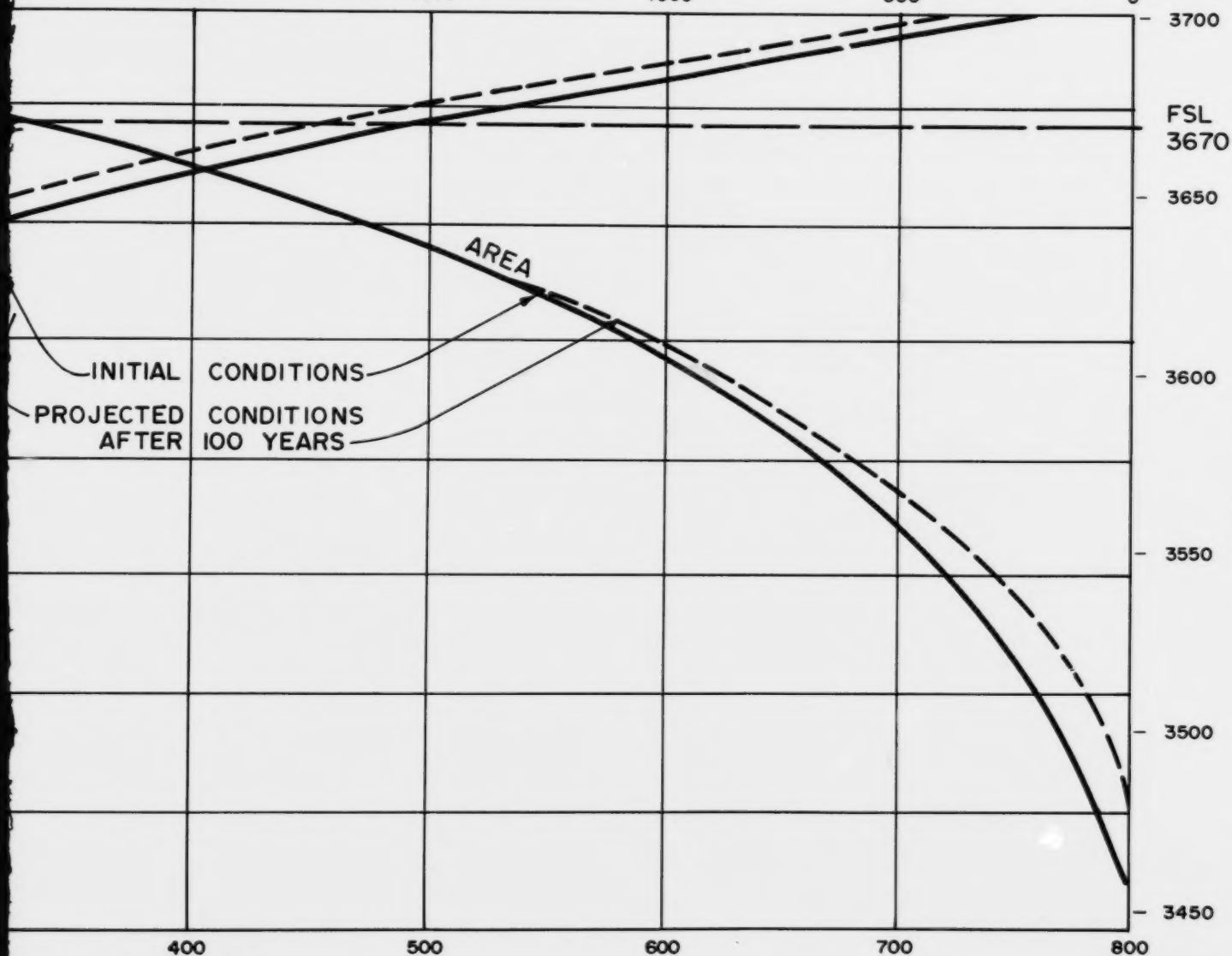
5000
|
(ha)
2000

4000
|
1500

3000
|
1000

2000
|
500

1000
|
0



INITIAL CONDITIONS

PROJECTED CONDITIONS AFTER 100 YEARS

AREA

ELEVATION (ft.)

RESERVOIR STORAGE VOLUME

400
(dam³ x 10³)

300
|

500
|

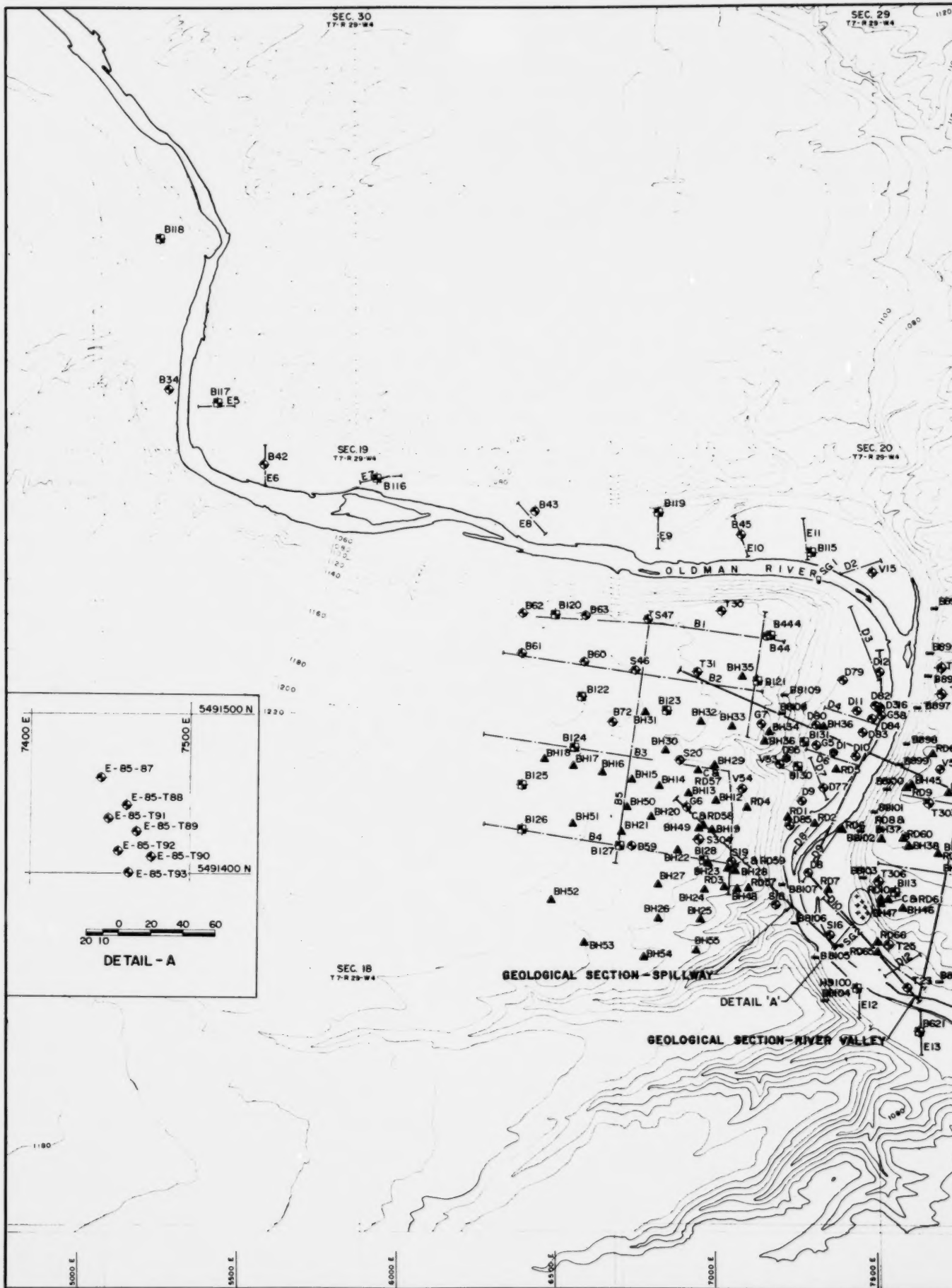
600
|

700
|

800
|

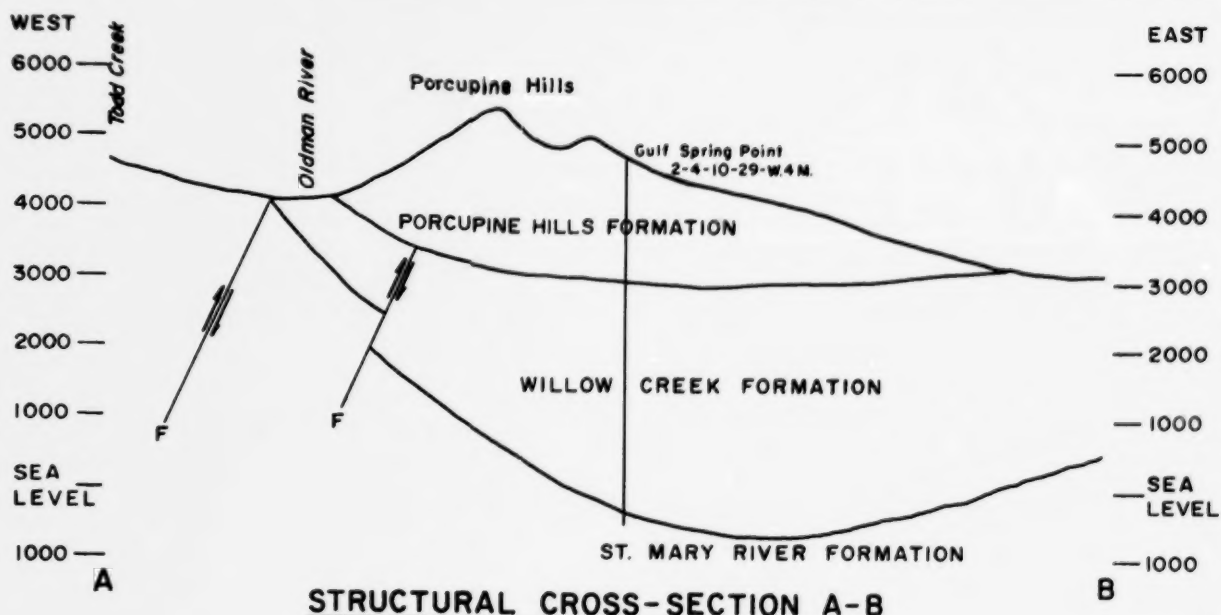
400
(ac.ft. x 10³)

		
OLDMAN RIVER DAM		
RIVER HYDRAULICS AND RESERVOIR STORAGE		
PLATE 5		



[illegible]

C



LEGEND:

- 1** PORCUPINE HILLS FORMATION: siltstone, sandstone, claystone
- 2** WILLOW CREEK FORMATION: claystone, siltstone, sandstone, occasional bentonite beds
- 3** Undifferentiated Jurassic-Cretaceous clastic rocks

Edge of disturbed belt. This marks the boundary between the Alberta Plain and the Rocky Mountain Foothills physiographic regions.

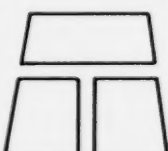
Geological boundary

Line of cross-section

Thrust fault (teeth indicate upthrust side)

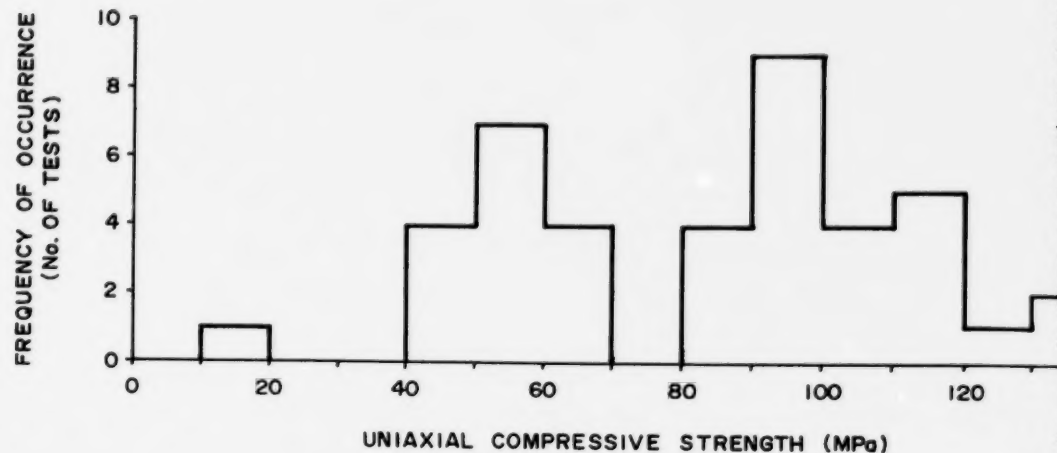
Alberta Syncline Axis

Note: Base map prepared from 1:250 000 N.T.S. map sheets 82/H and 82/G, Surveys and Mapping Branch, Department of Energy, Mines and Resources.

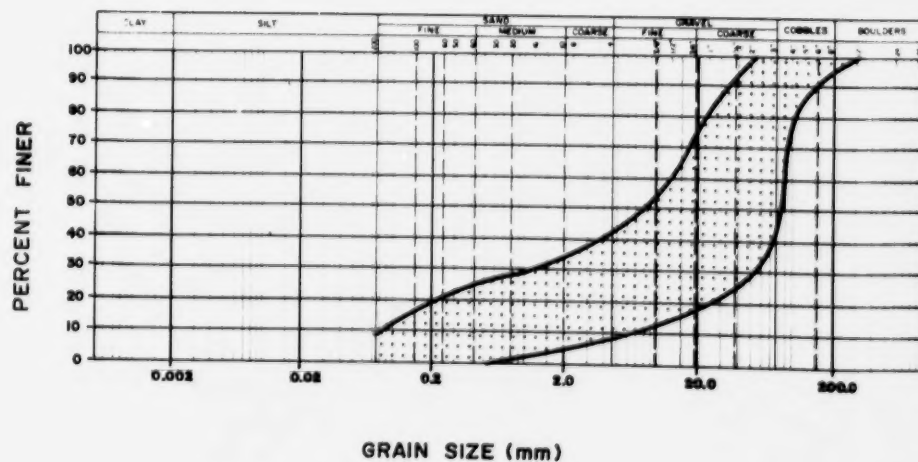
	U.M.A. ENGINEERING LTD.		DRAWN	L.E.N. / B.R.
	REGIONAL BEDROCK GEOLOGY		TRACED	
			DATE	NOV., 1985
			APPROVED	
	OLDMAN RIVER DAM		PINCHER CREEK, ALBERTA	SCALE
THURBER CONSULTANTS LTD., Geotechnical Engineers				PLATE 7

RANGE (MPa) AVERAGE

VB	12 - 116	79
AB	44 - 132	84

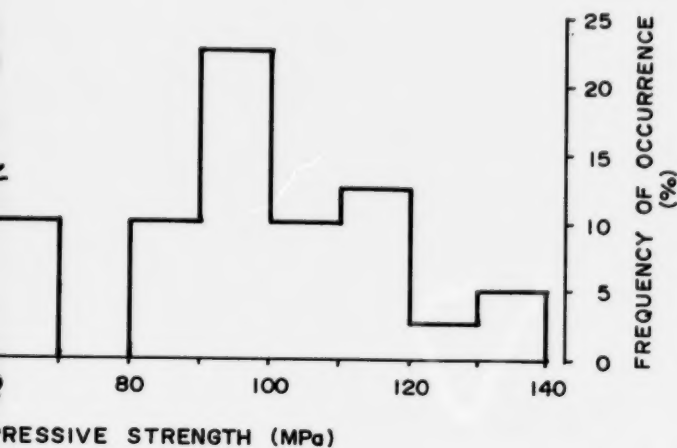


SANDSTONE



RIVER VALLEY ALLUVIUM

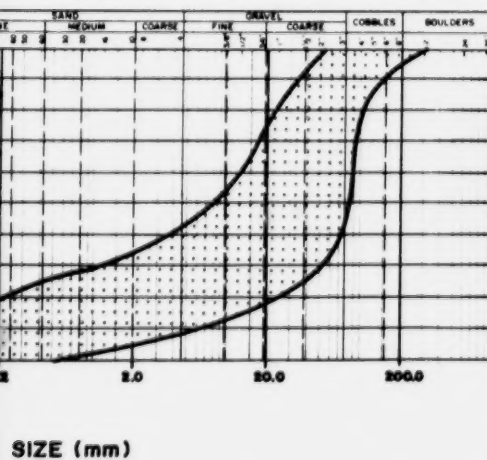
	RANGE (MPa)	AVERAGE (MPa)	No. OF TESTS
VB	12 - 116	79	15
AB	44 - 132	84	26



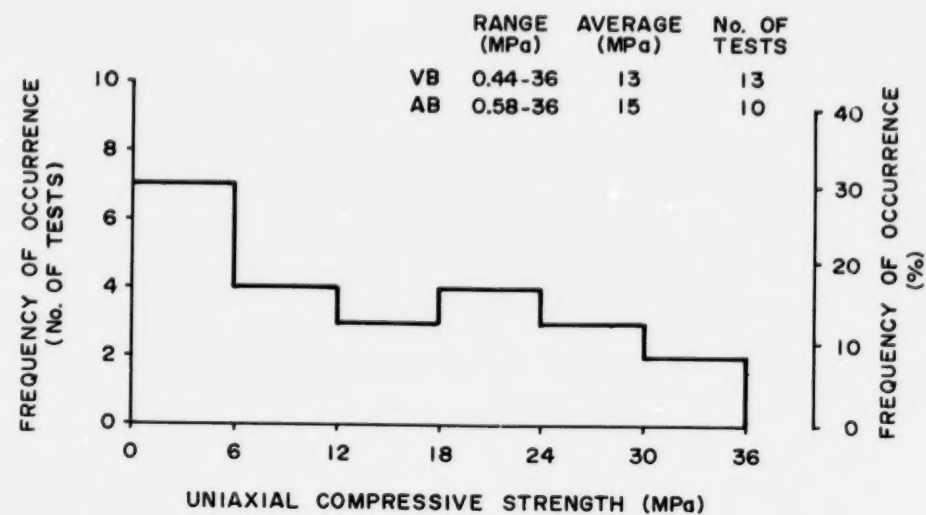
STONE

LEGEND

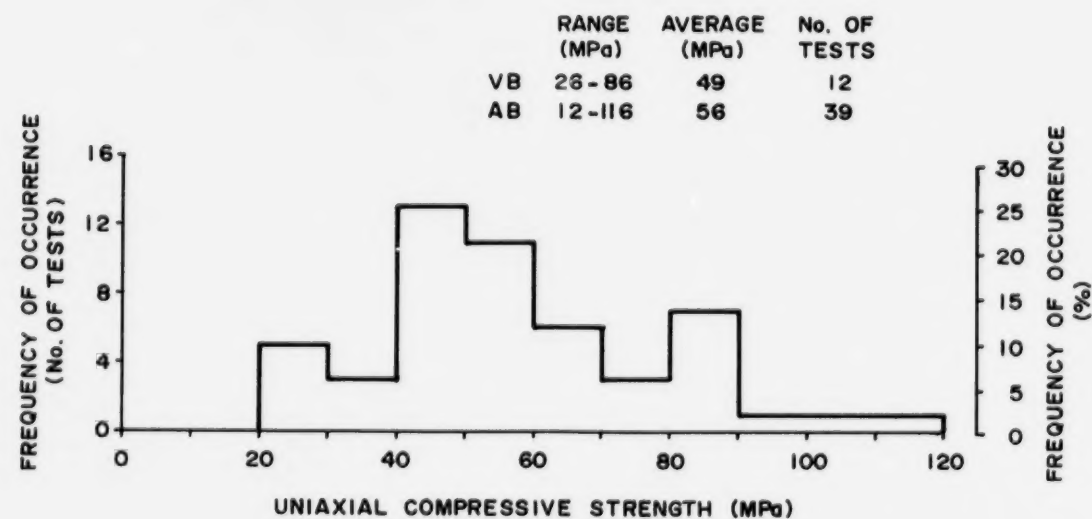
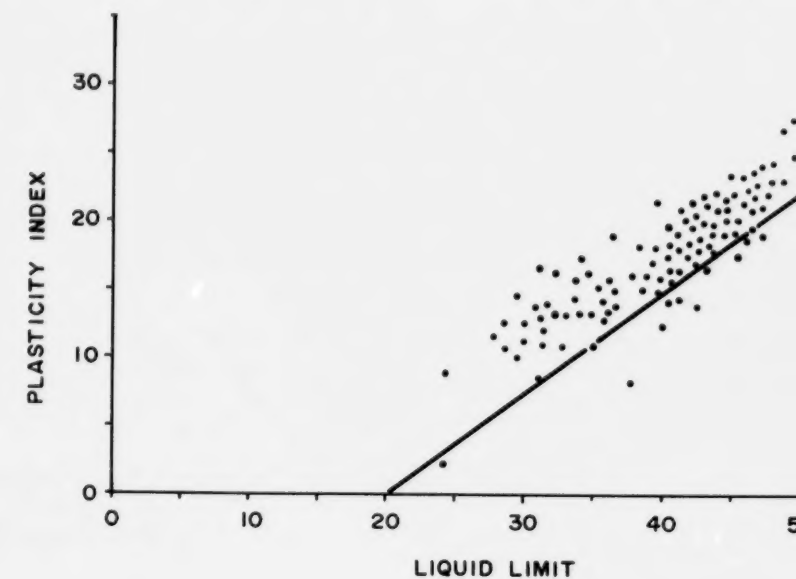
VB - VALLEY BOTTOM
AB - ABUTMENTS



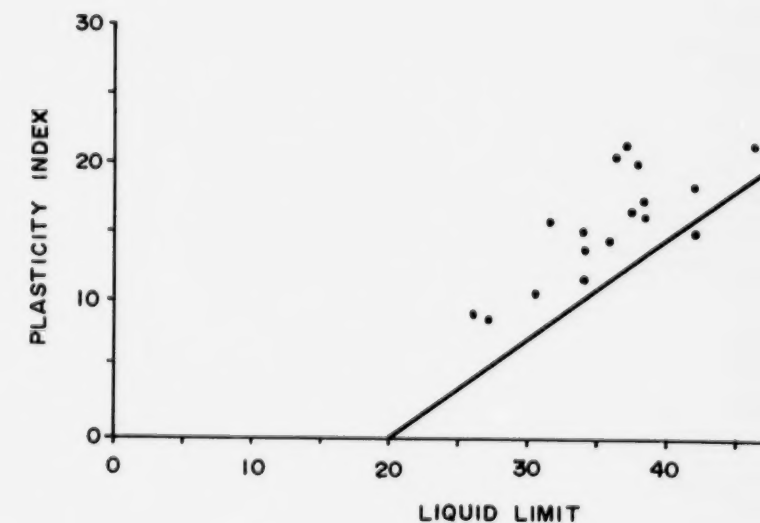
LEY ALLUVIUM

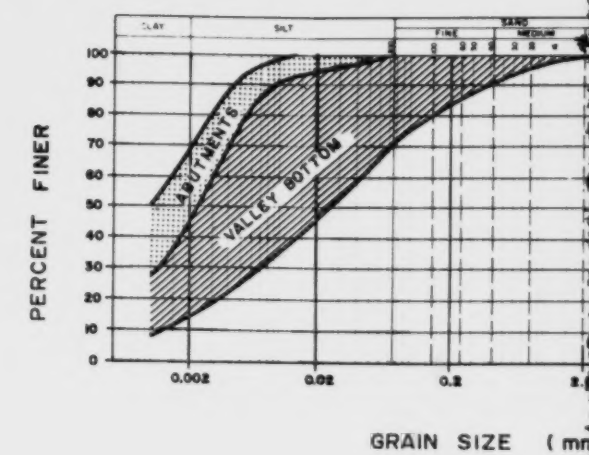
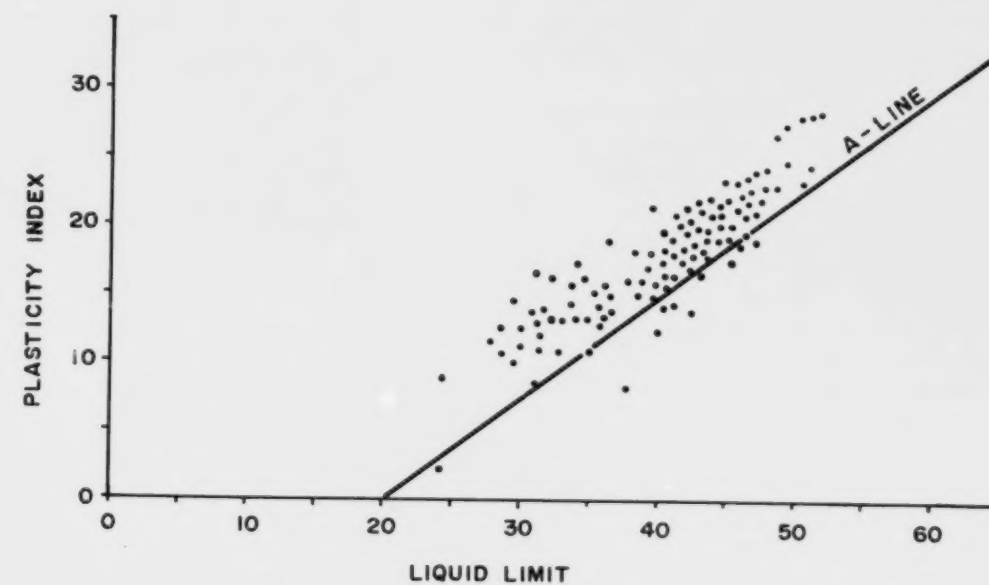
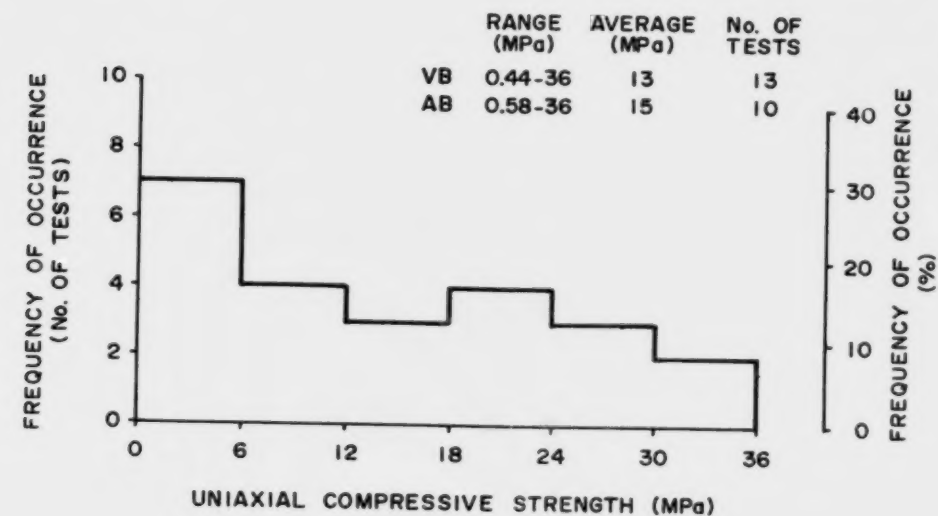


CLAYSTONE

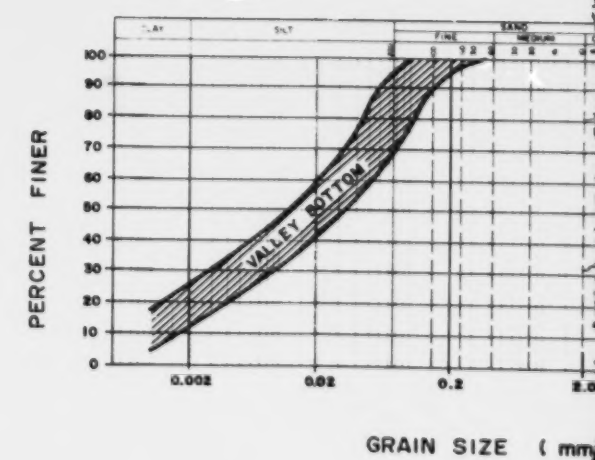
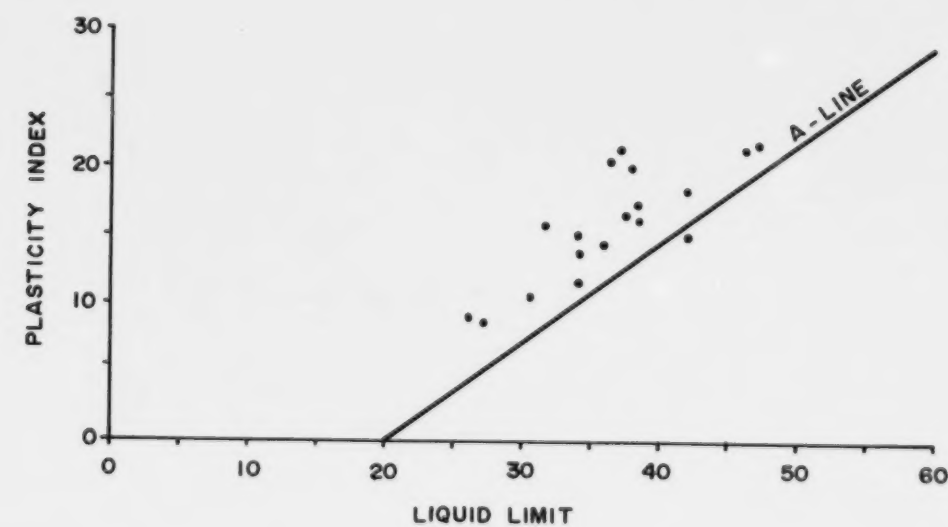
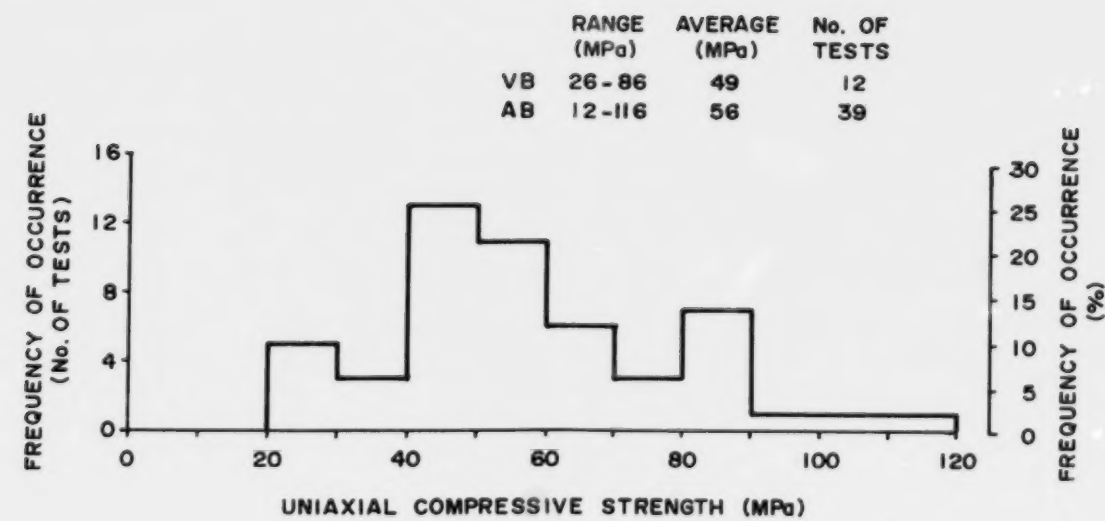


SILTSTONE

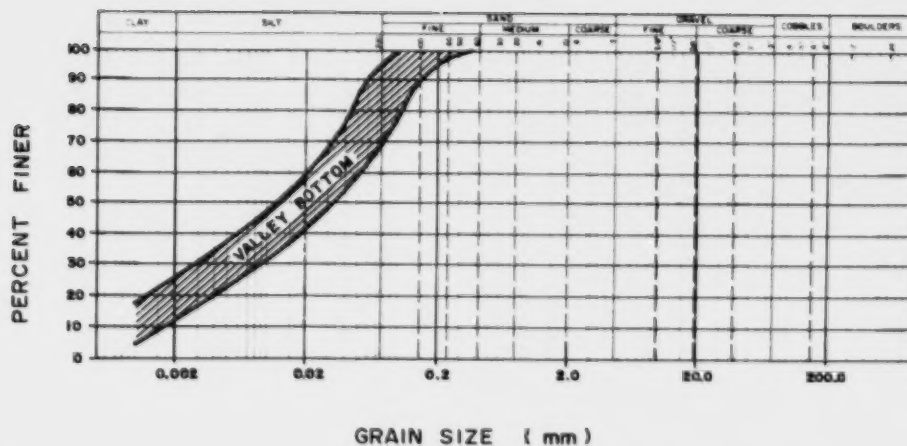
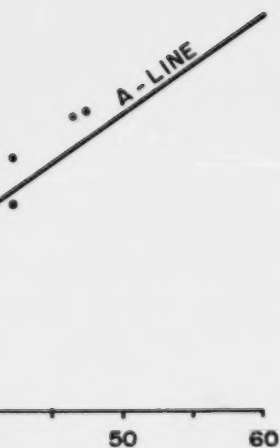
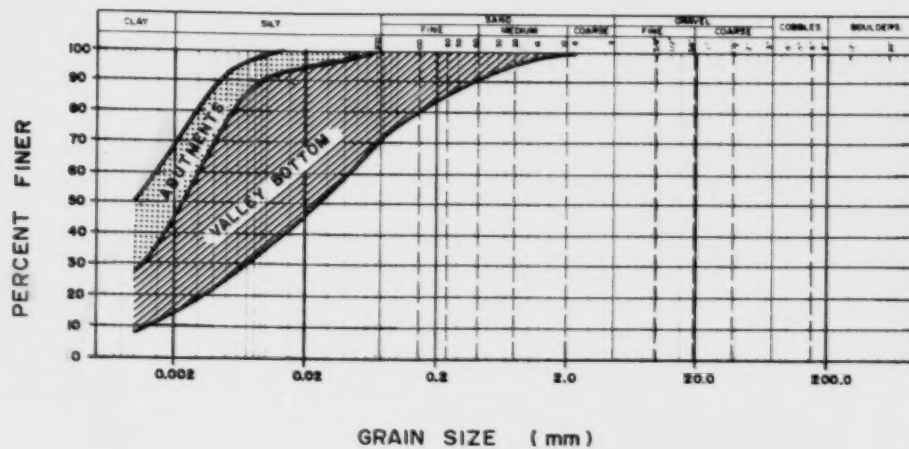


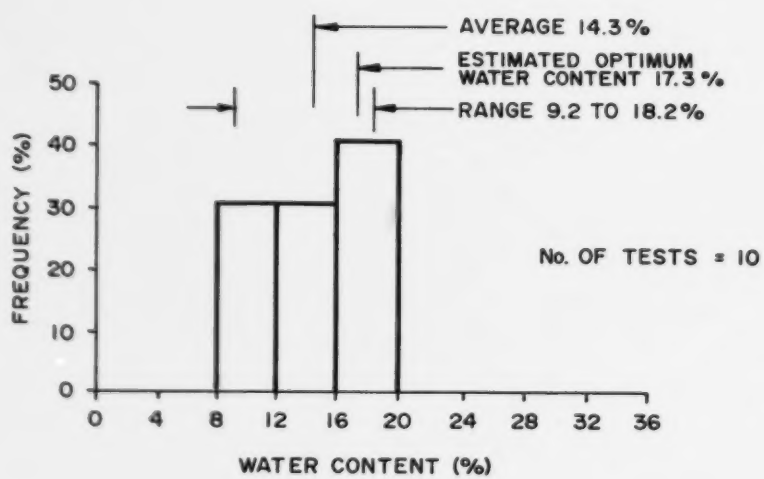
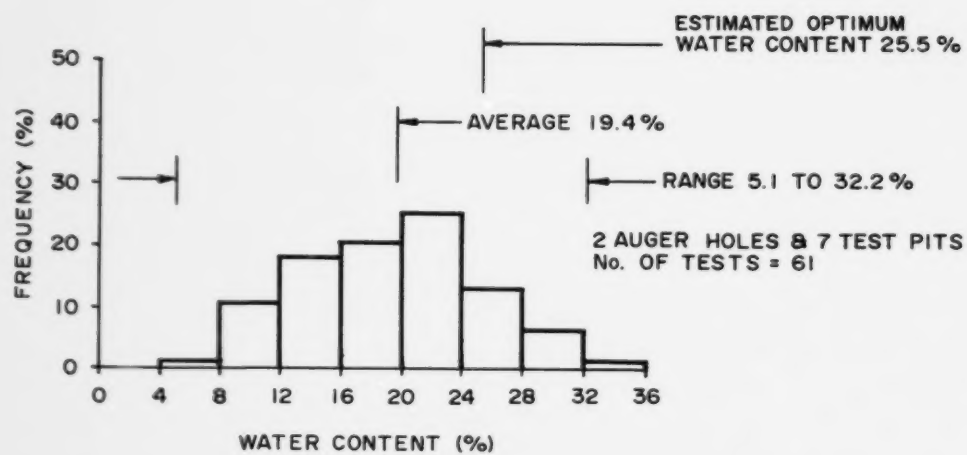


CLAYSTONE



SILTSTONE



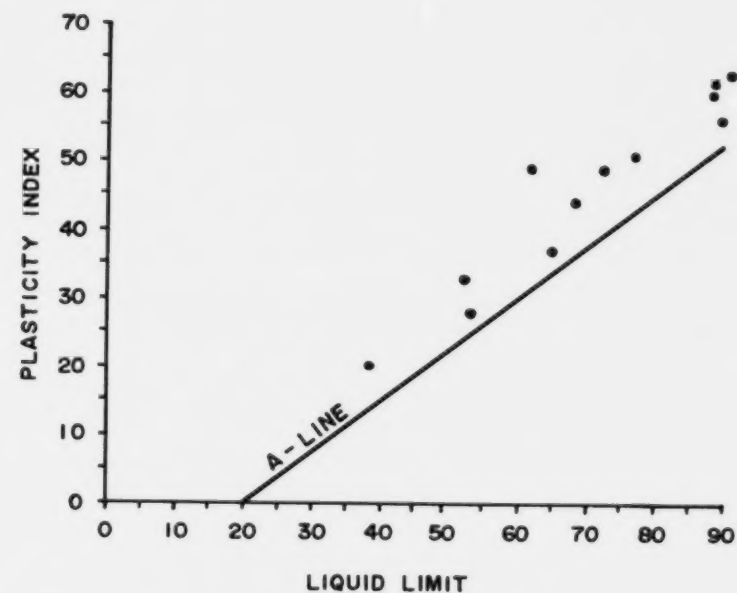
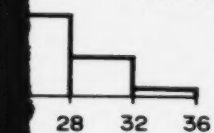


ESTIMATED OPTIMUM
WATER CONTENT 25.5 %

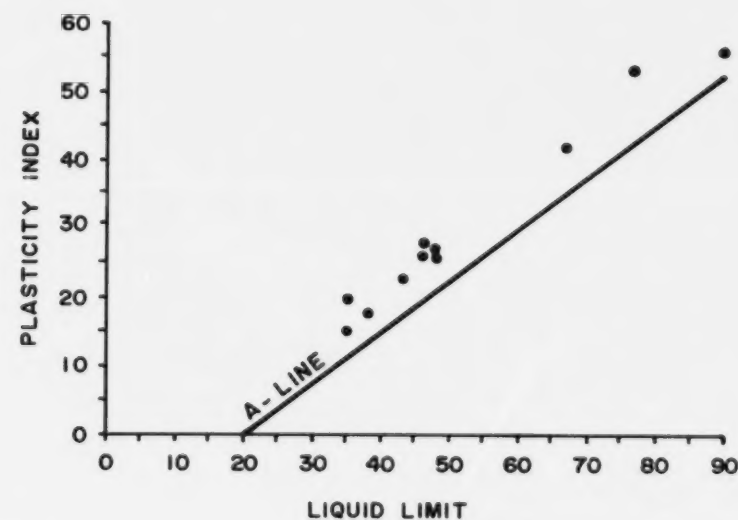
AVERAGE 19.4 %

RANGE 5.1 TO 32.2 %

2 AUGER HOLES & 7 TEST PITS
No. OF TESTS = 61



GLACIO-LACUSTRINE CLAY



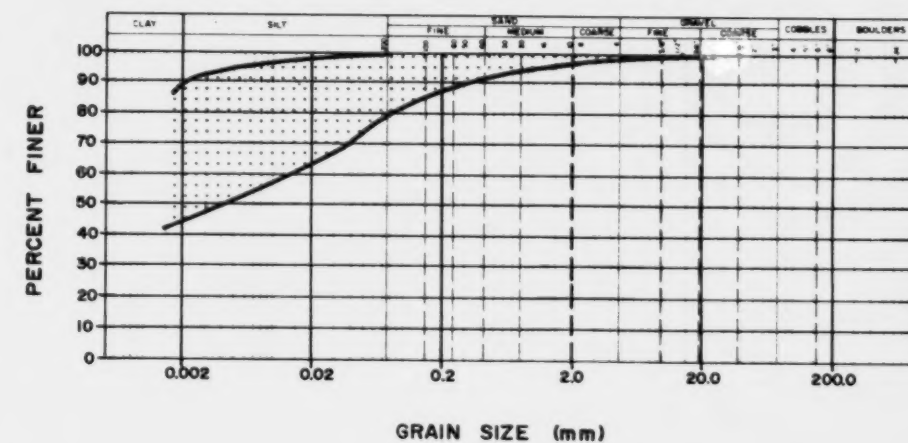
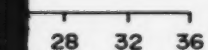
GLACIAL TILL

AVERAGE 14.3 %

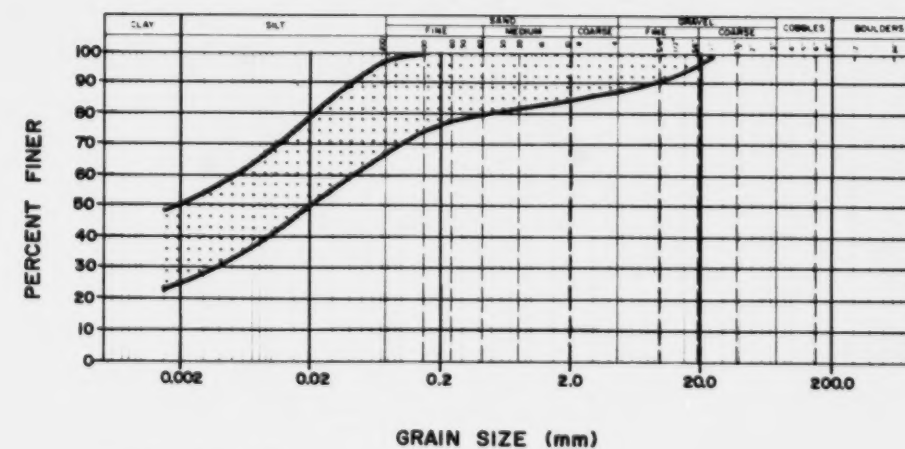
ESTIMATED OPTIMUM
WATER CONTENT 17.3 %

RANGE 9.2 TO 18.2 %

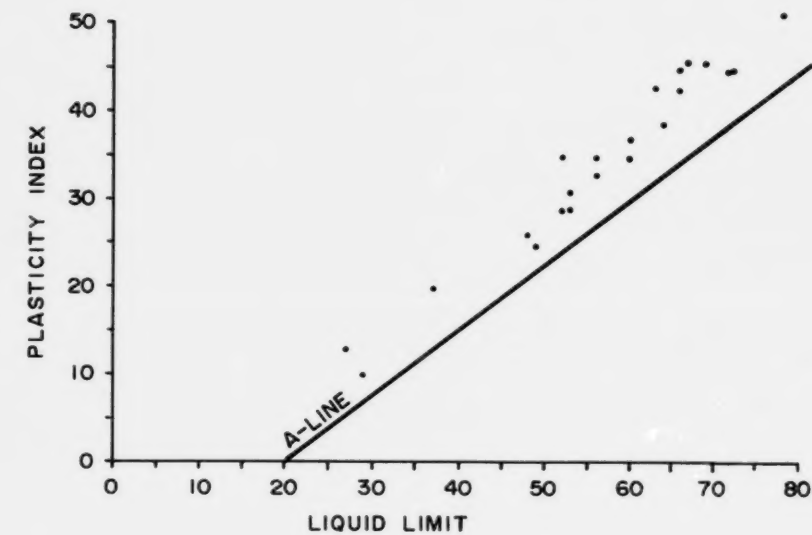
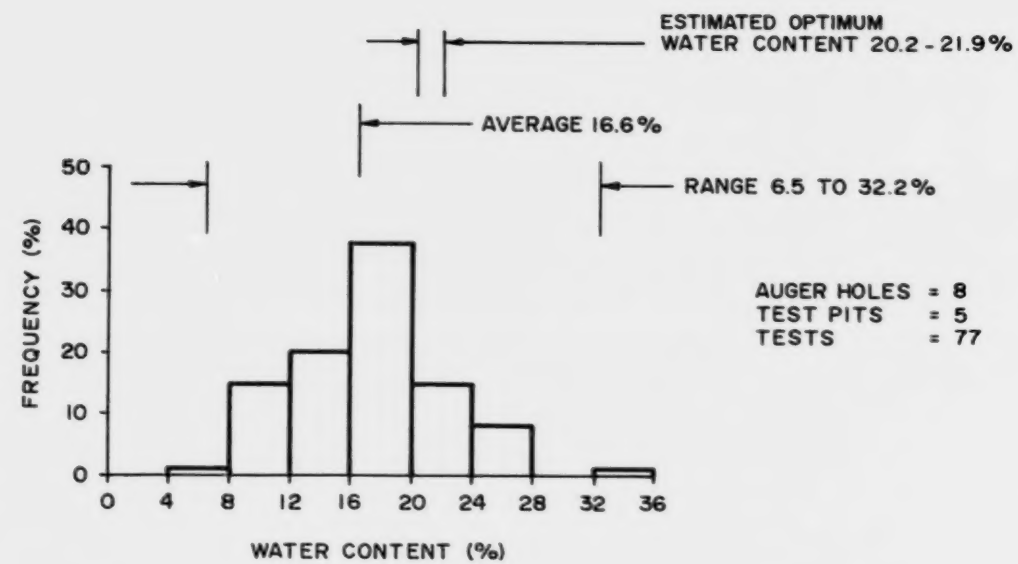
No. OF TESTS = 10



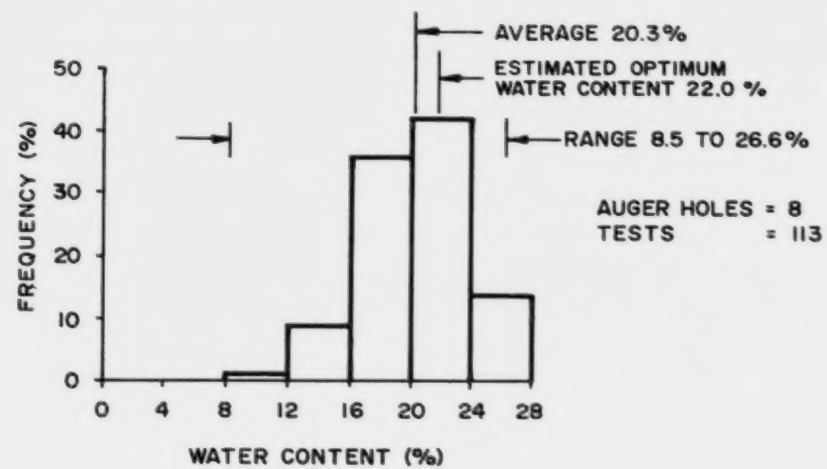
TOTAL 10 TESTS



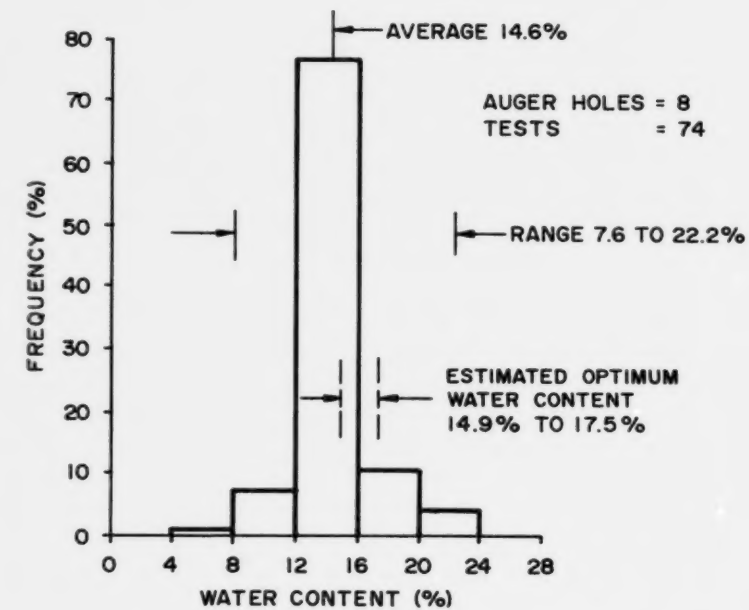
TOTAL 10 TESTS



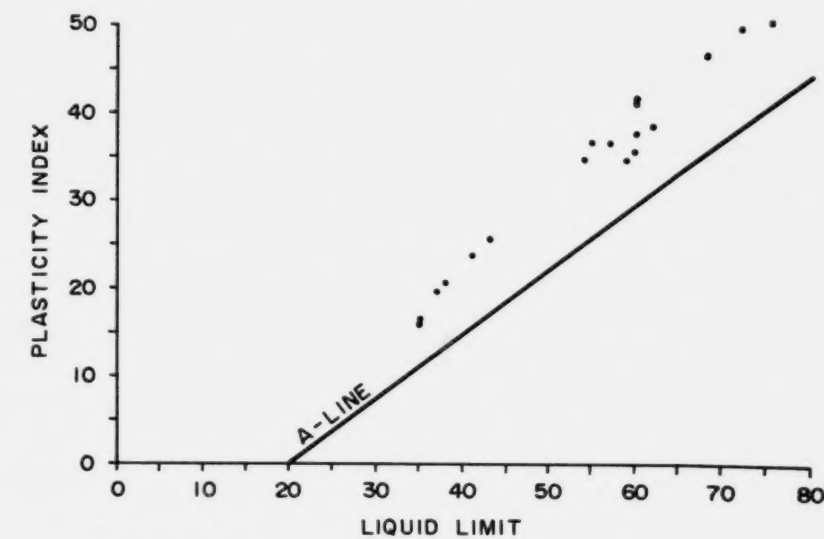
GLACIO-LACUSTRINE CLAY



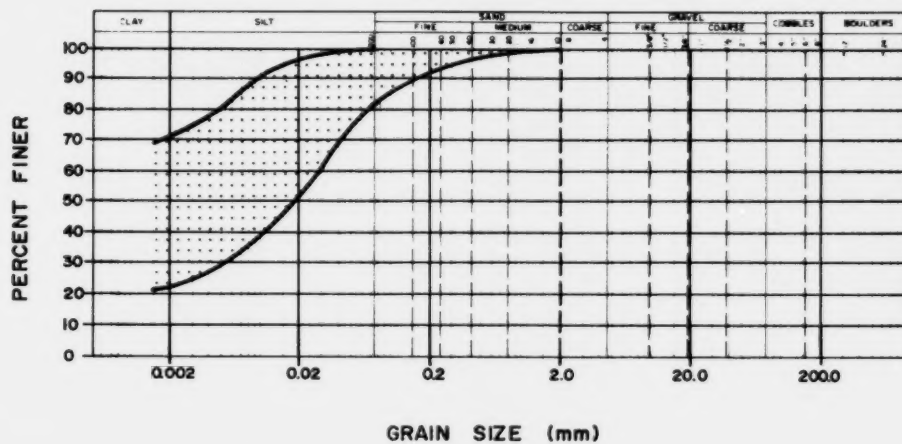
CLAY TILL (HIGHLY PLASTIC)



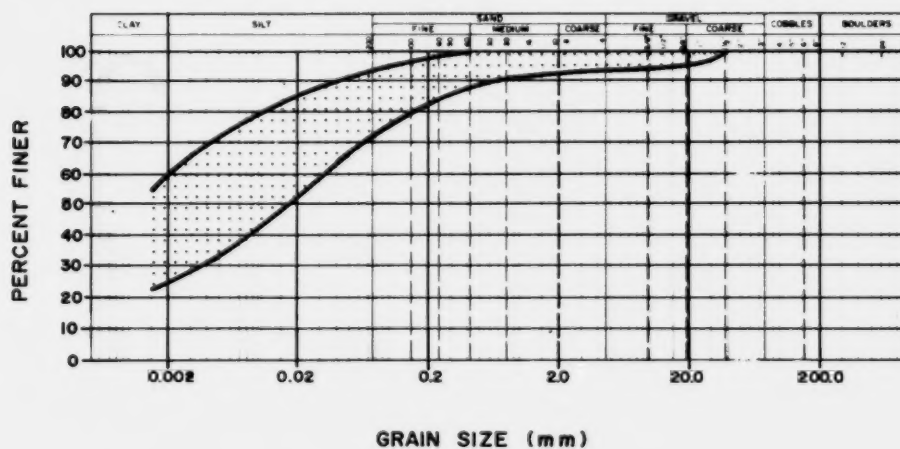
SILTY CLAY TILL (MEDIUM PLASTIC)



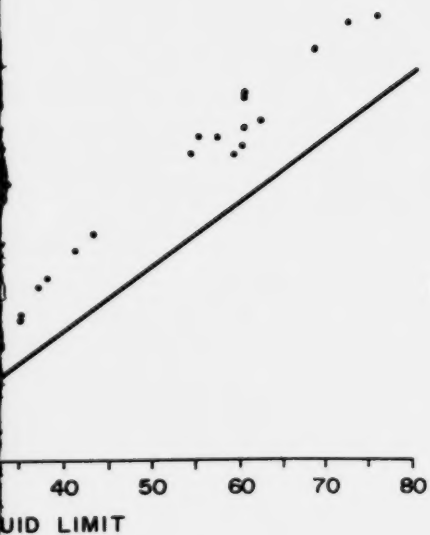
GLACIAL TILL



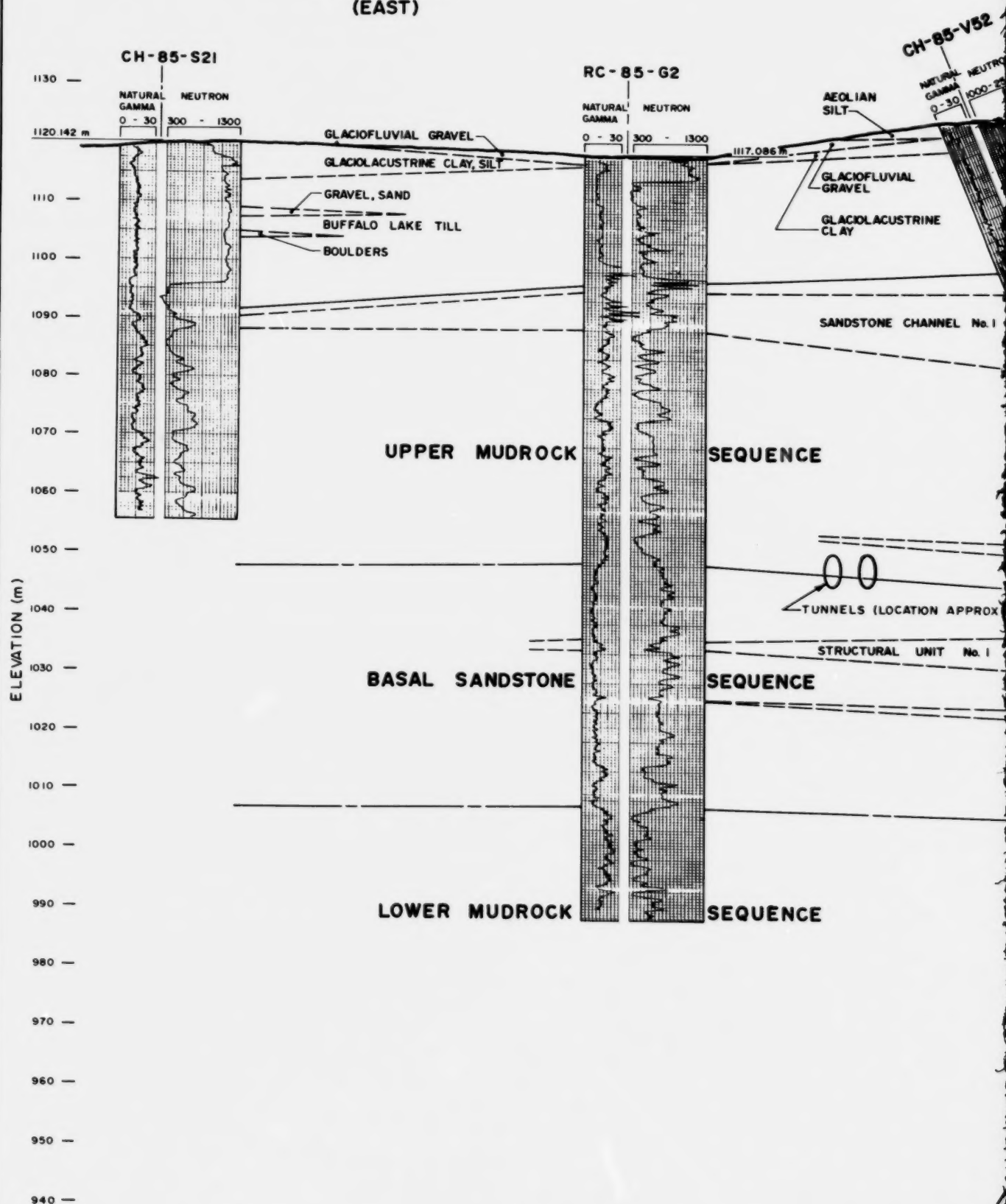
TOTAL 24 TESTS



TOTAL 32 TESTS

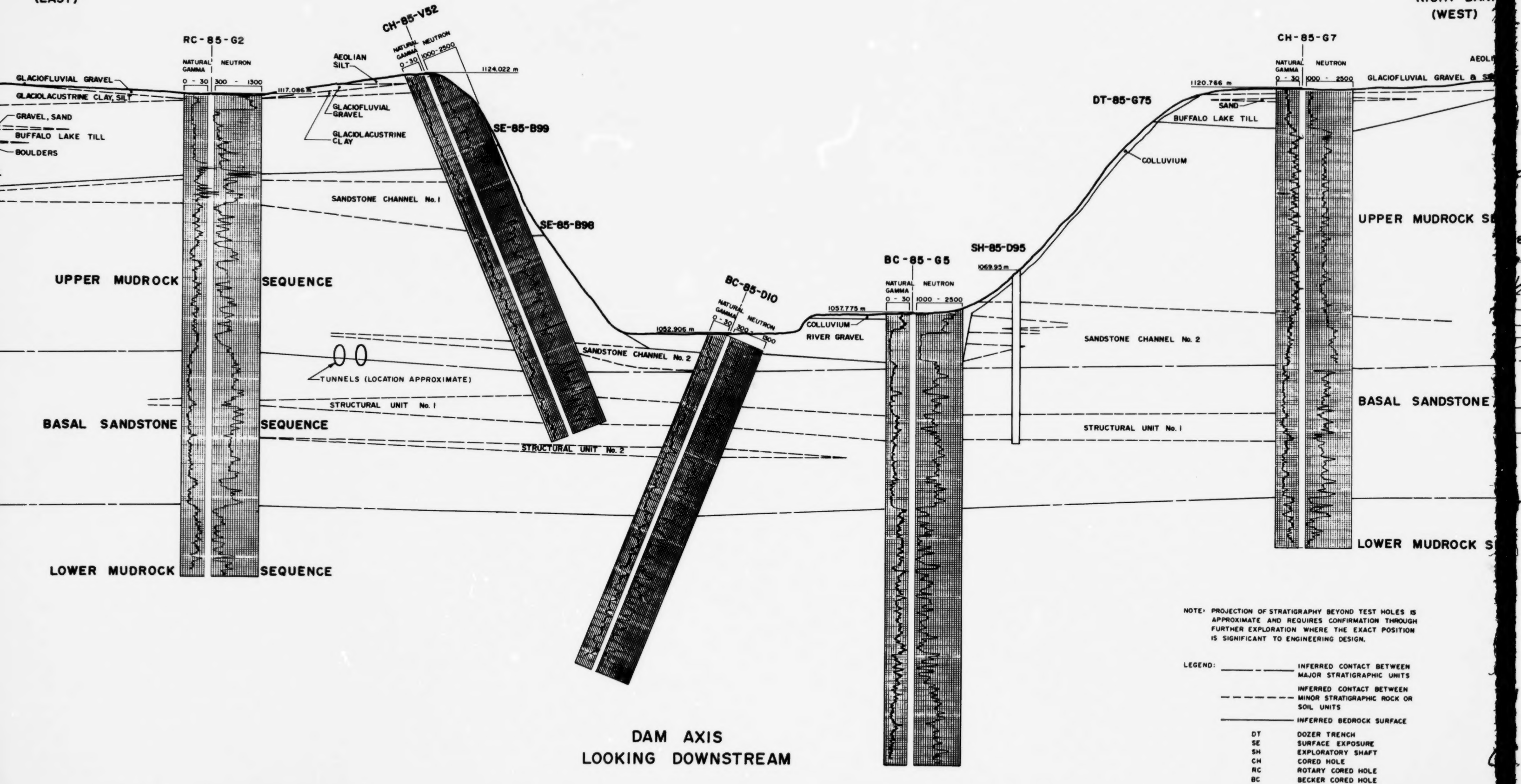


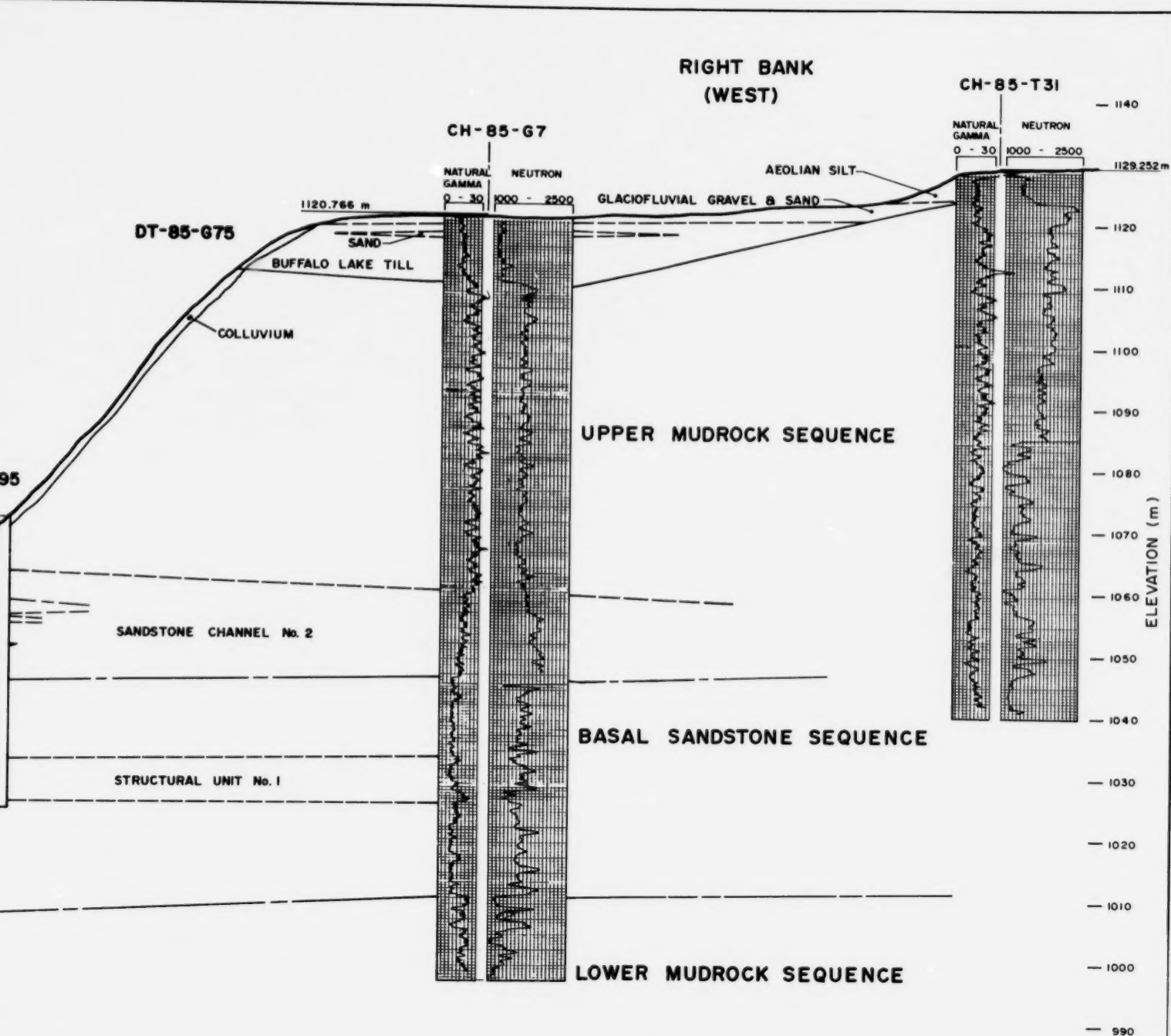
LEFT BANK (EAST)



LEFT BANK
(EAST)

RIGHT BANK
(WEST)





NOTE: PROJECTION OF STRATIGRAPHY BEYOND TEST HOLES IS APPROXIMATE AND REQUIRES CONFIRMATION THROUGH FURTHER EXPLORATION WHERE THE EXACT POSITION IS SIGNIFICANT TO ENGINEERING DESIGN.

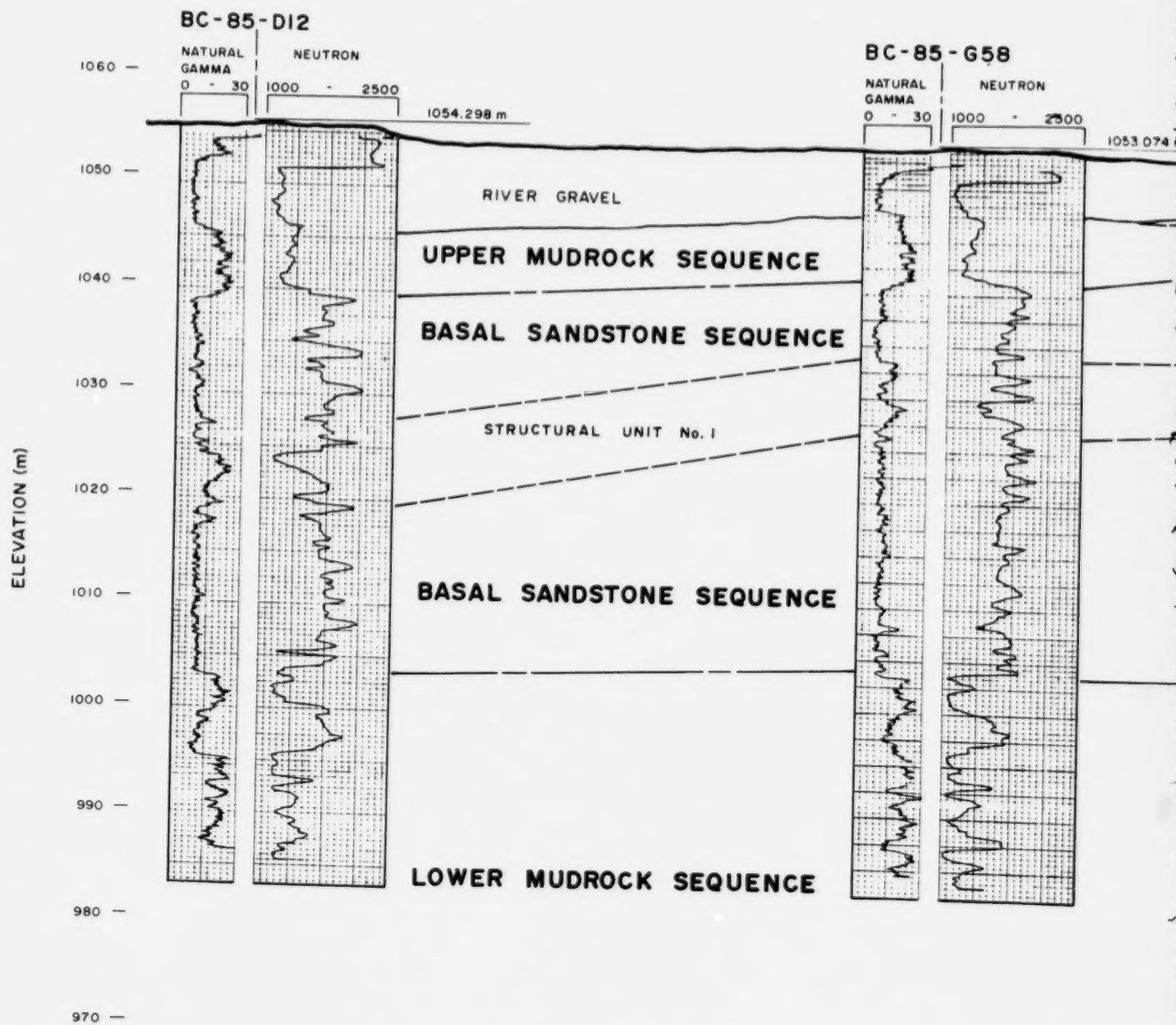
LEGEND:

- INFERRED CONTACT BETWEEN MAJOR STRATIGRAPHIC UNITS
- - - - - INFERRED CONTACT BETWEEN MINOR STRATIGRAPHIC ROCK OR SOIL UNITS
- INFERRED BEDROCK SURFACE

DT DOZER TRENCH
SE SURFACE EXPOSURE
SH EXPLORATORY SHAFT
CH CORED HOLE
RC ROTARY CORED HOLE
BC BECKER CORED HOLE

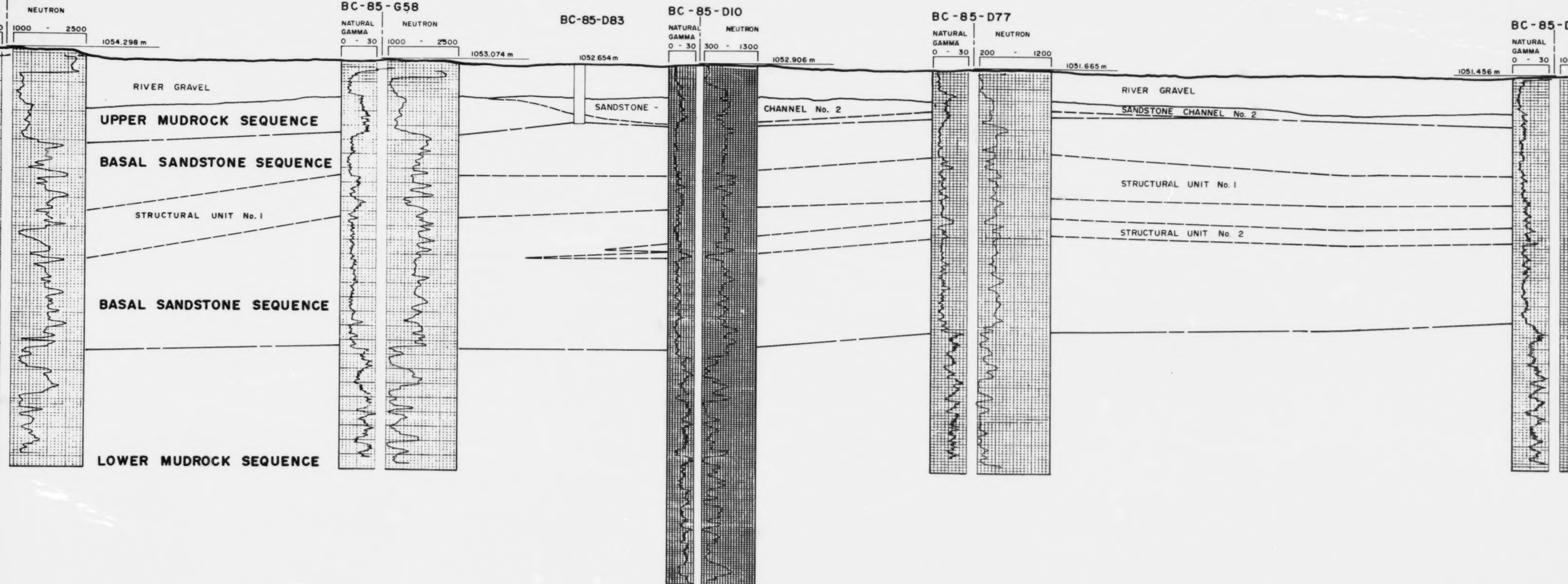
OLDMAN RIVER DAM	
GEOLOGICAL SECTION DAM AXIS	
PLATE 11	

UPSTREAM
(NORTH)



STREAM
(NORTH)

5-D12



RIVER VALLEY
LOOKING EAST

-D77

NEUTRON

200 - 1200

1051.665 m

BC-85-D8

NATURAL
GAMMA
0 - 30

NEUTRON

1000 - 2500

1051.456 m

BC-85-S16

NATURAL
GAMMA
0 - 30

NEUTRON

1000 - 2500

1052.558 m

DOWN
(SO

BC-85

NATURAL
GAMMA
0 - 30

RIVER GRAVEL

SANDSTONE CHANNEL No. 2

STRUCTURAL UNIT No. 1

STRUCTURAL UNIT No. 2

SANDSTONE CHANNEL No. 2

UPPER MUDROCK SEQUENCE

BASAL SANDSTONE SEQUENCE

STRUCTURAL UNIT No. 1

STRUCTURAL UNIT No. 2

BASAL SANDSTONE SEQUENCE

RIVER GRAVEL

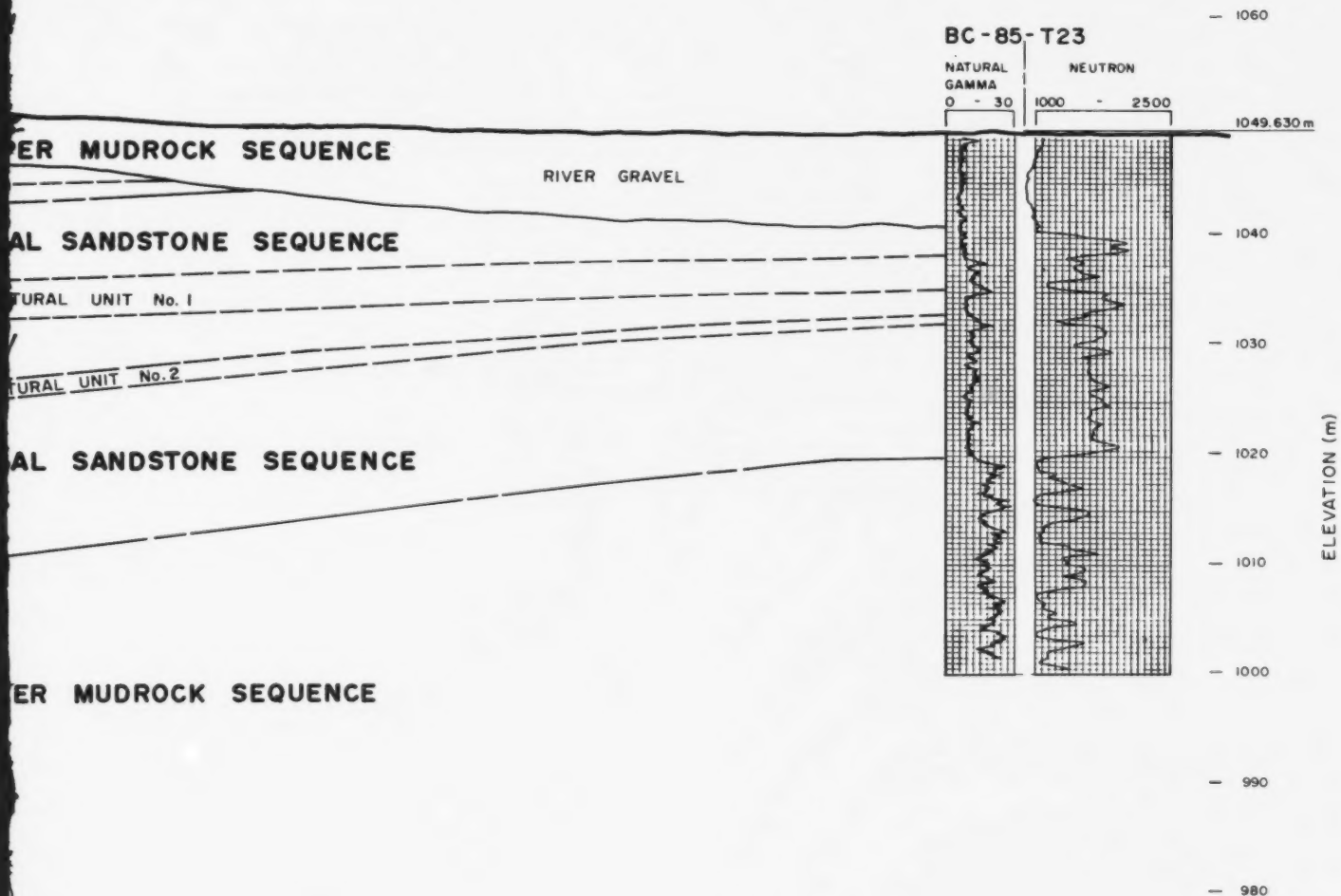
LOWER MUDROCK SEQUENCE

RIVER VALLEY
LOOKING EAST

NOTE: PROJECTION OF STRATIGRAPHY BEYOND TEST HOLES IS APPROXIMATE AND REQUIRES CONFIRMATION THROUGH FURTHER EXPLORATION WHERE THE EXACT POSITION IS SIGNIFICANT TO ENGINEERING DESIGN.

LEGEND: — — — — — INFERRED CONTACT BETWEEN MAJOR STRATIGRAPHIC UNITS
- - - - - INFERRED CONTACT BETWEEN MINOR STRATIGRAPHIC ROCK OR SOIL UNITS
— — — — — INFERRED BEDROCK SURFACE
DT DOZER TRENCH
SE SURFACE EXPOSURE
SH EXPLORATORY SHAFT
CH CORED HOLE
RC ROTARY CORED HOLE
BC BECKER CORED HOLE

**DOWNSTREAM
(SOUTH)**



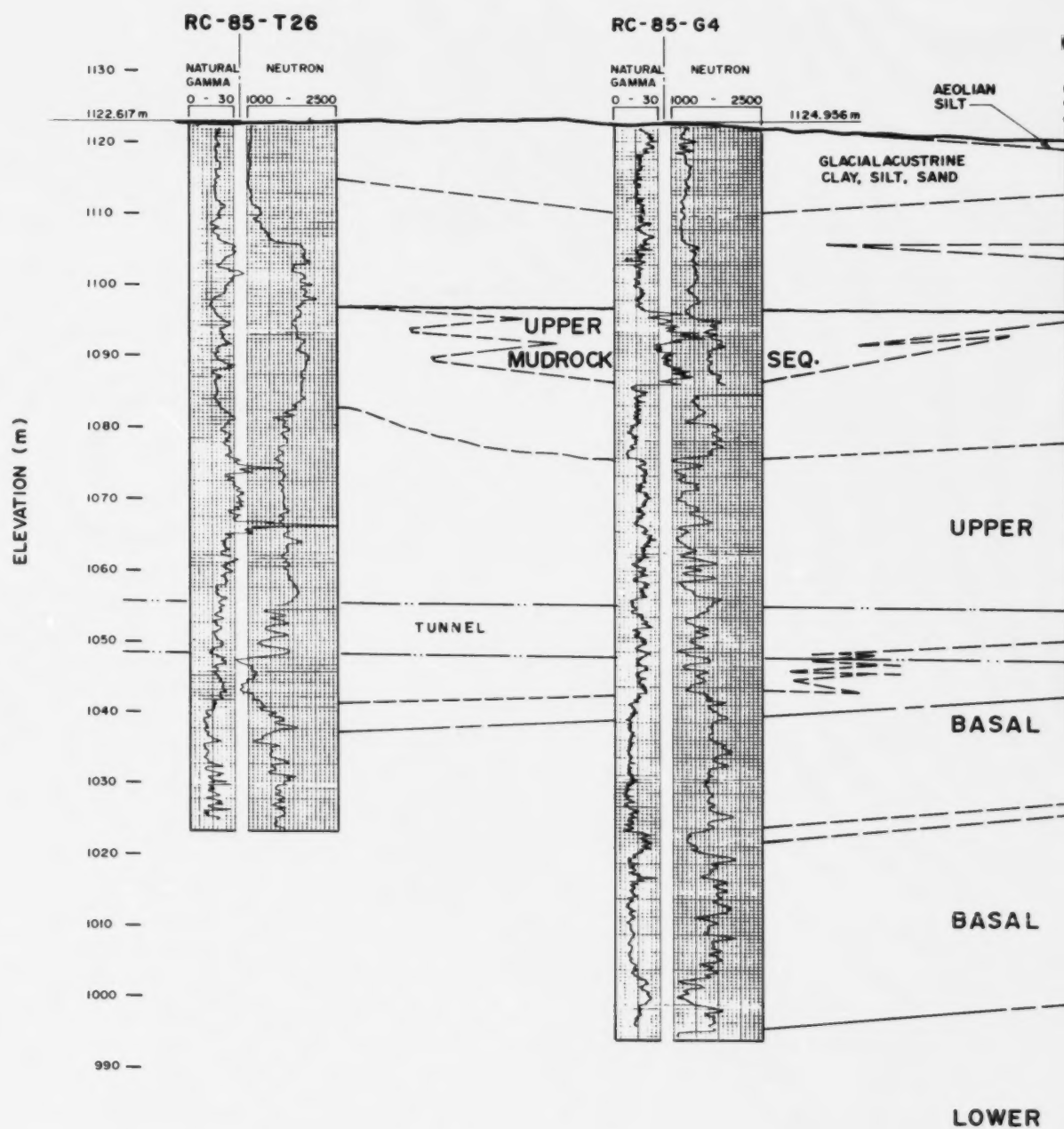
NOTE: PROJECTION OF STRATIGRAPHY BEYOND TEST HOLES IS APPROXIMATE AND REQUIRES CONFIRMATION THROUGH FURTHER EXPLORATION WHERE THE EXACT POSITION IS SIGNIFICANT TO ENGINEERING DESIGN.

LEGEND: --- INFERRED CONTACT BETWEEN MAJOR STRATIGRAPHIC UNITS
 - - - - - INFERRED CONTACT BETWEEN MINOR STRATIGRAPHIC ROCK OR SOIL UNITS
 ——— INFERRED BEDROCK SURFACE
 DT DOZER TRENCH
 SE SURFACE EXPOSURE
 SH EXPLORATORY SHAFT
 CH CORED HOLE
 RC ROTARY CORED HOLE
 BC BECKER CORED HOLE

OLDMAN RIVER DAM
 GEOLOGICAL SECTION
 RIVER VALLEY

PLATE 12

UPSTREAM (NORTH)

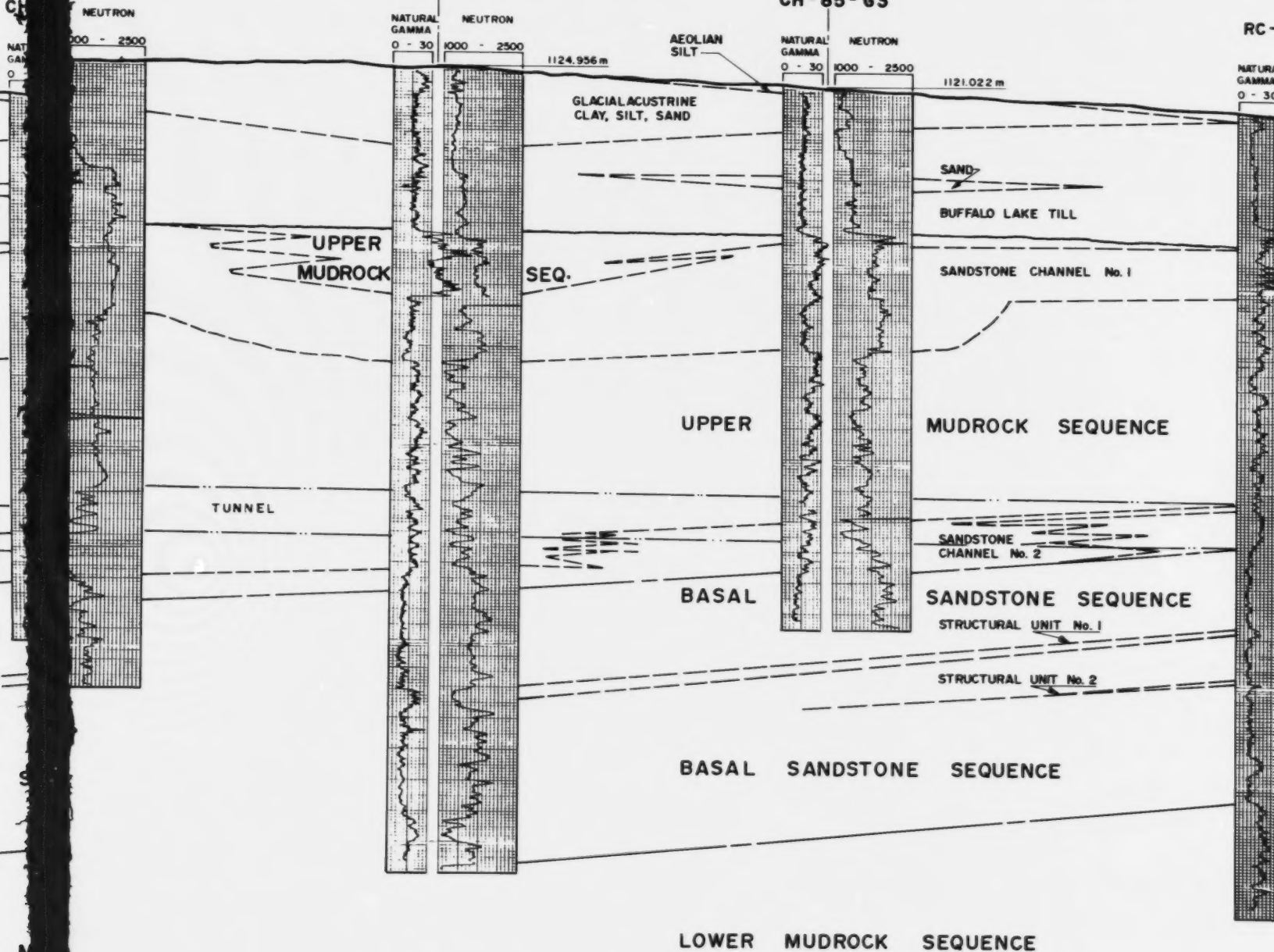


TEAM
(TH)

-T26

RC-85-G4

CH-85-G3



RC-

NATURAL
GAMMA
0 - 30

LE

5- G2

NEUTRON

300 - 1300

1117.086 m

AEOLIAN
SILT

GLACIOFLUVIAL GRAVEL

BUFFALO LAKE TILL

CH-85-T24

NATURAL
GAMMA
0 - 30

NEUTRON

1000 - 2500

1093.142 m

SE-85-T23

SE-85-B8110

DOWN

(SC

BC

NATU
GAMMA
0 - 3

1049.08m

SANDSTONE CHANNEL NO. 2.

TUNNEL
OUTLET

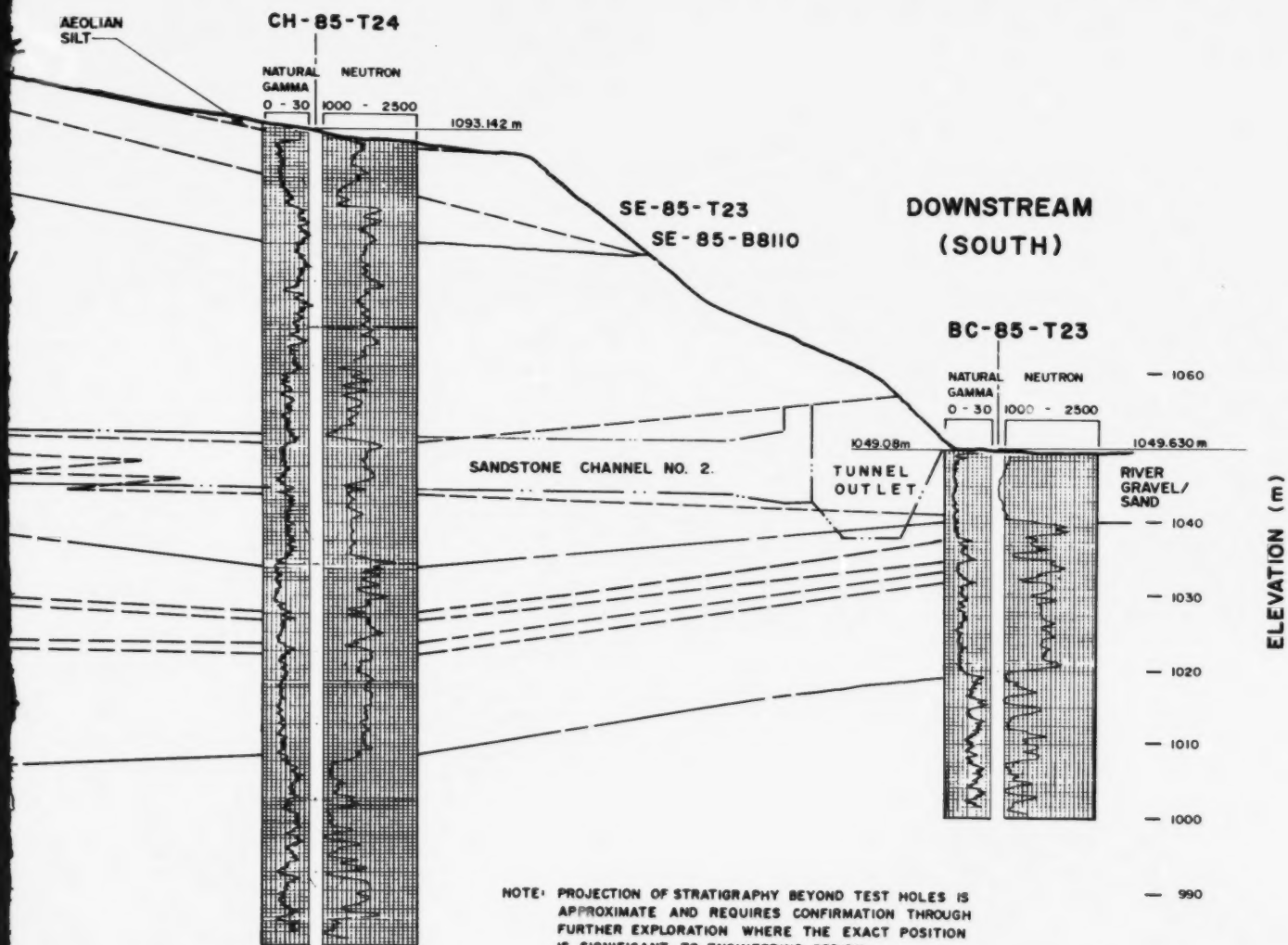
NOTE: PROJECTION OF STRATIGRAPHY BEYOND TEST HOLES IS APPROXIMATE AND REQUIRES CONFIRMATION THROUGH FURTHER EXPLORATION WHERE THE EXACT POSITION IS SIGNIFICANT TO ENGINEERING DESIGN.

TUNNEL OUTLINE IS PROJECTED ONTO THE SECTION. THE ROCK SEQUENCES INTERSECTED ALONG THE TUNNEL WILL DIFFER FROM THOSE SHOWN.

LEGEND:
 ——— INFERRED CONTACT BETWEEN MAJOR STRATIGRAPHIC UNITS
 - - - - - INFERRED CONTACT BETWEEN MINOR STRATIGRAPHIC ROCK OR SOIL UNITS
 ——— INFERRED BEDROCK SURFACE

DT DOZER TRENCH
 SE SURFACE EXPOSURE
 SH EXPLORATORY SHAFT
 CH CORED HOLE
 RC ROTARY CORED HOLE
 BC BECKER CORED HOLE

TUNNEL
T BANK



NOTE: PROJECTION OF STRATIGRAPHY BEYOND TEST HOLES IS APPROXIMATE AND REQUIRES CONFIRMATION THROUGH FURTHER EXPLORATION WHERE THE EXACT POSITION IS SIGNIFICANT TO ENGINEERING DESIGN.

TUNNEL OUTLINE IS PROJECTED ONTO THE SECTION. THE ROCK SEQUENCES INTERSECTED ALONG THE TUNNEL WILL DIFFER FROM THOSE SHOWN.

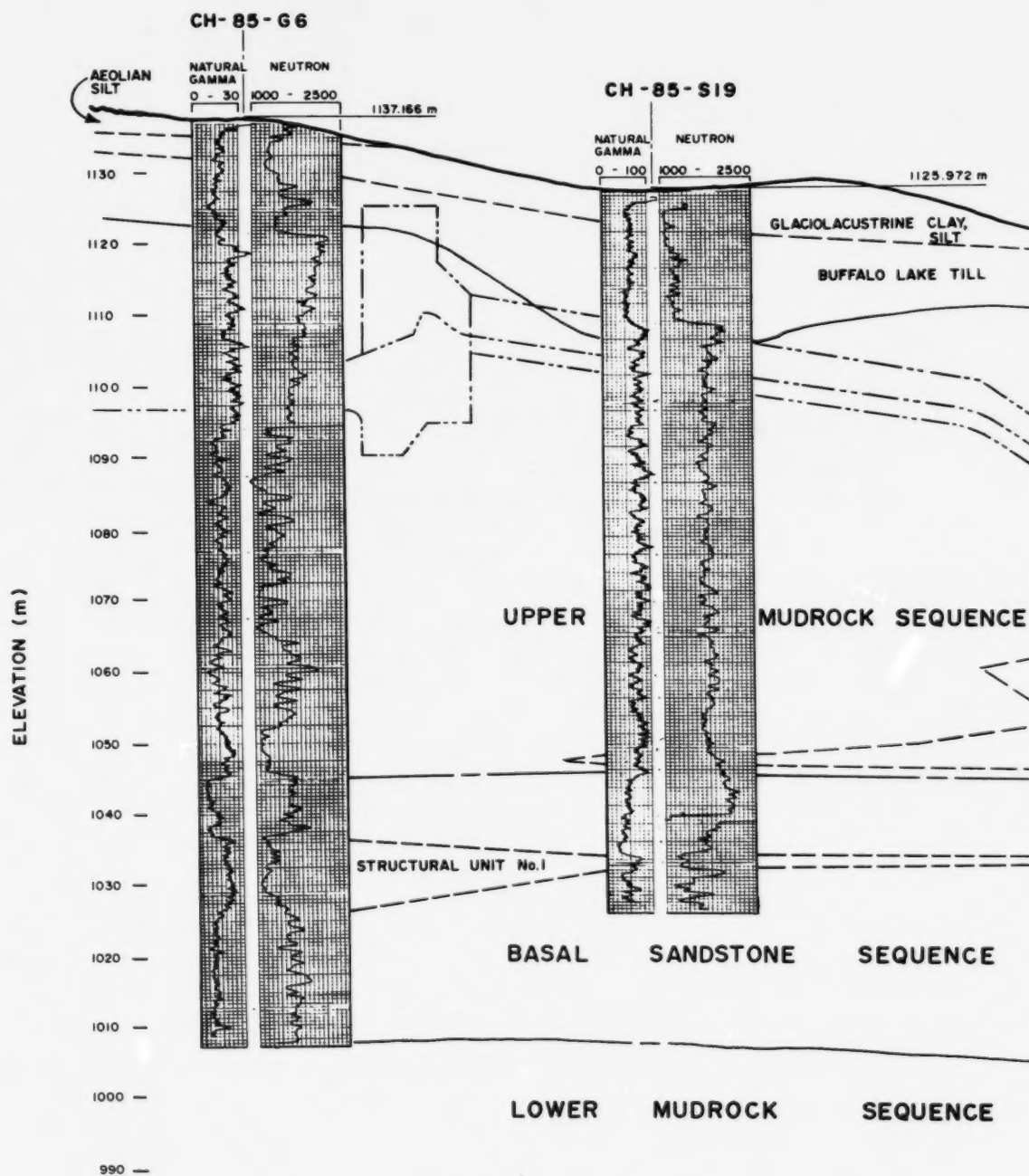
LEGEND:

- INFERRED CONTACT BETWEEN MAJOR STRATIGRAPHIC UNITS
- INFERRED CONTACT BETWEEN MINOR STRATIGRAPHIC ROCK OR SOIL UNITS
- INFERRED BEDROCK SURFACE

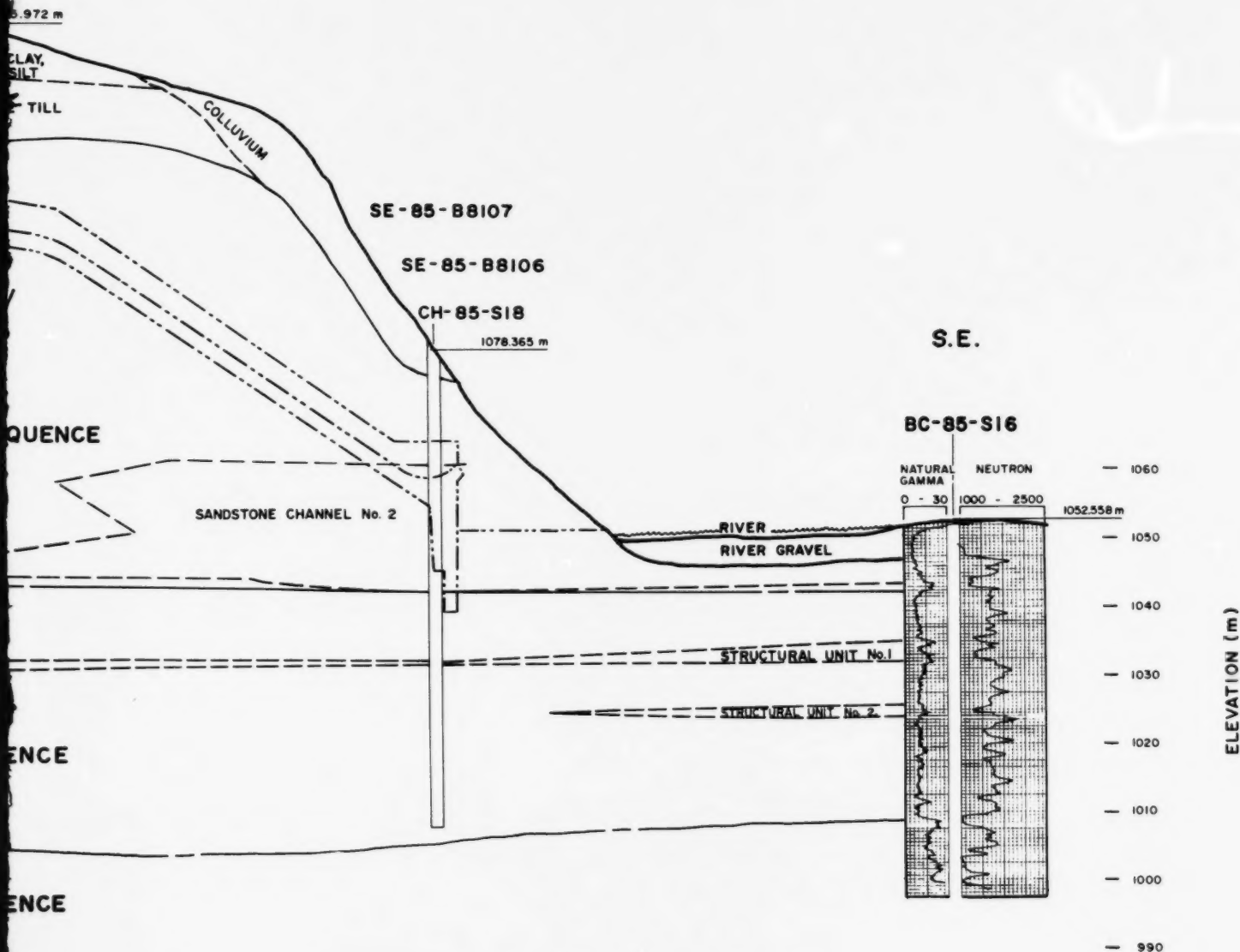
- DT DOZER TRENCH
- SE SURFACE EXPOSURE
- SH EXPLORATORY SHAFT
- CH CORED HOLE
- RC ROTARY CORED HOLE
- BC BECKER CORED HOLE

OLDMAN RIVER DAM
GEOLOGICAL SECTION TUNNEL
PLATE 13

N.W.



NOTE: PRO
AP
FUR
IS
SP
TH
SP



NOTE: PROJECTION OF STRATIGRAPHY BEYOND TEST HOLES IS APPROXIMATE AND REQUIRES CONFIRMATION THROUGH FURTHER EXPLORATION WHERE THE EXACT POSITION IS SIGNIFICANT TO ENGINEERING DESIGN.

SPILLWAY OUTLINE IS PROJECTED ONTO THE SECTION. THE ROCK SEQUENCES INTERSECTED ALONG THE SPILLWAY WILL DIFFER FROM THOSE SHOWN.

LEGEND:

——— INFERRED CONTACT BETWEEN MAJOR STRATIGRAPHIC UNITS

- - - - - INFERRED CONTACT BETWEEN MINOR STRATIGRAPHIC ROCK OR SOIL UNITS

————— INFERRED BEDROCK SURFACE

DT DOZER TRENCH

SE SURFACE EXPOSURE

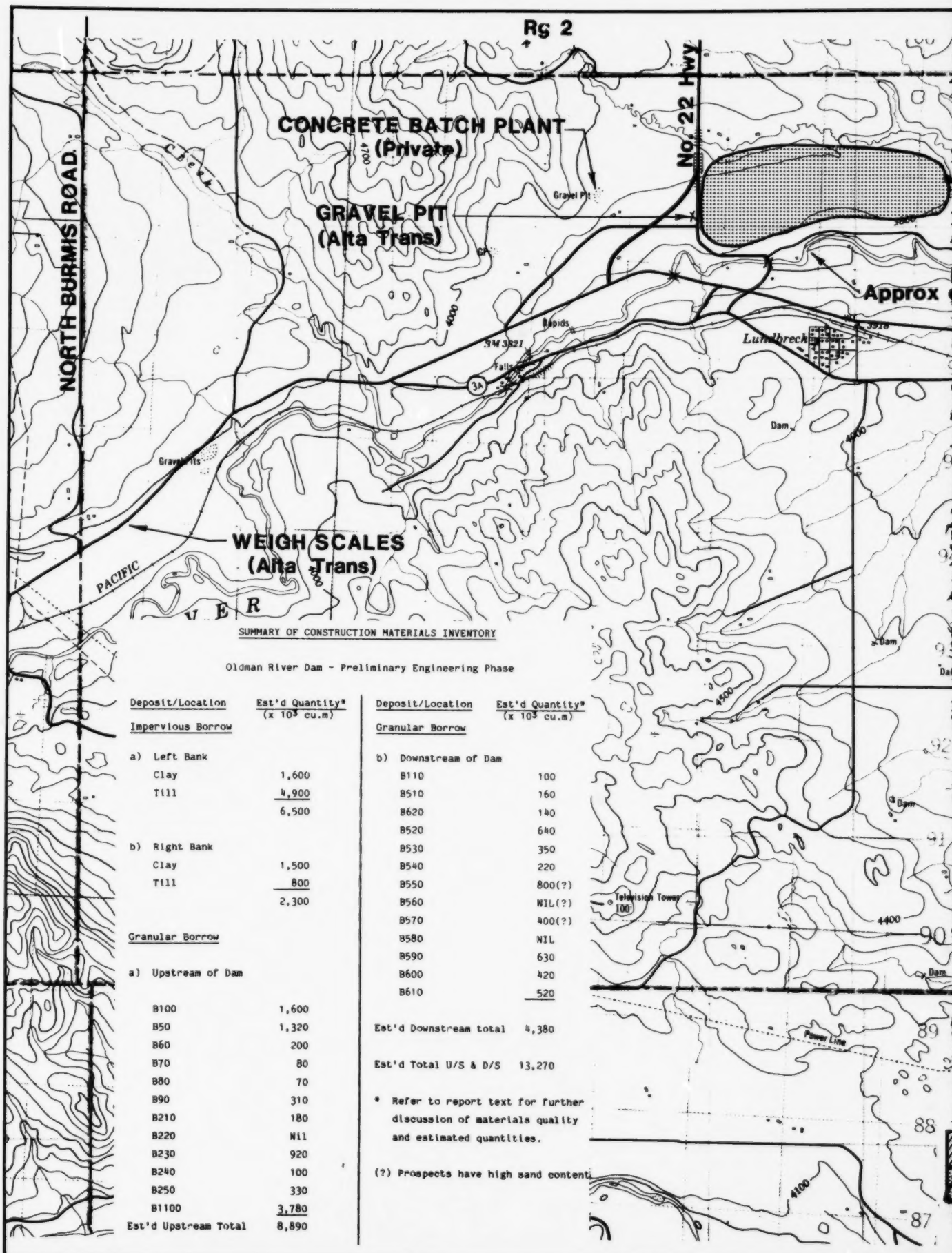
SH EXPLORATORY SHAFT

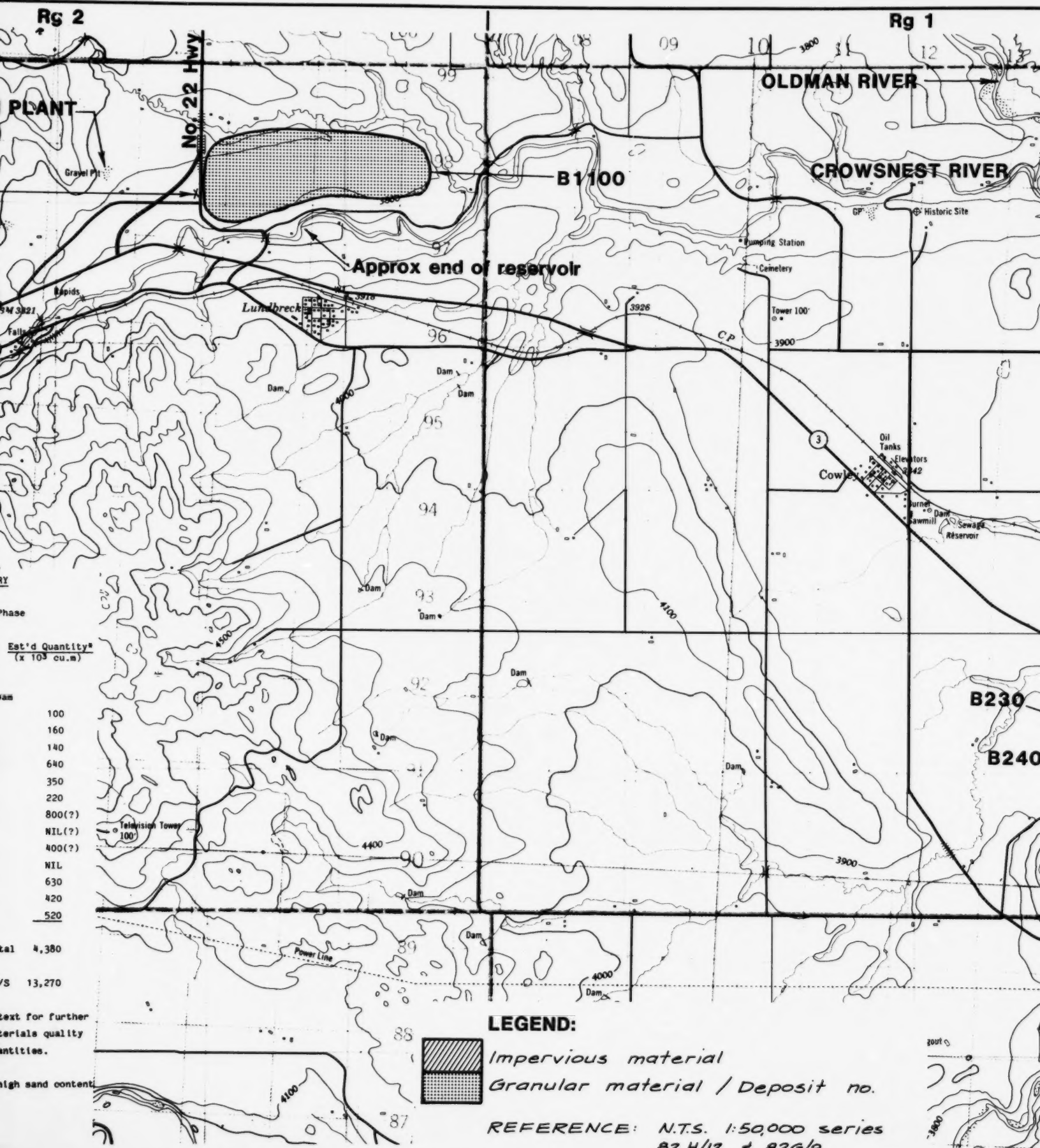
CH CORED HOLE

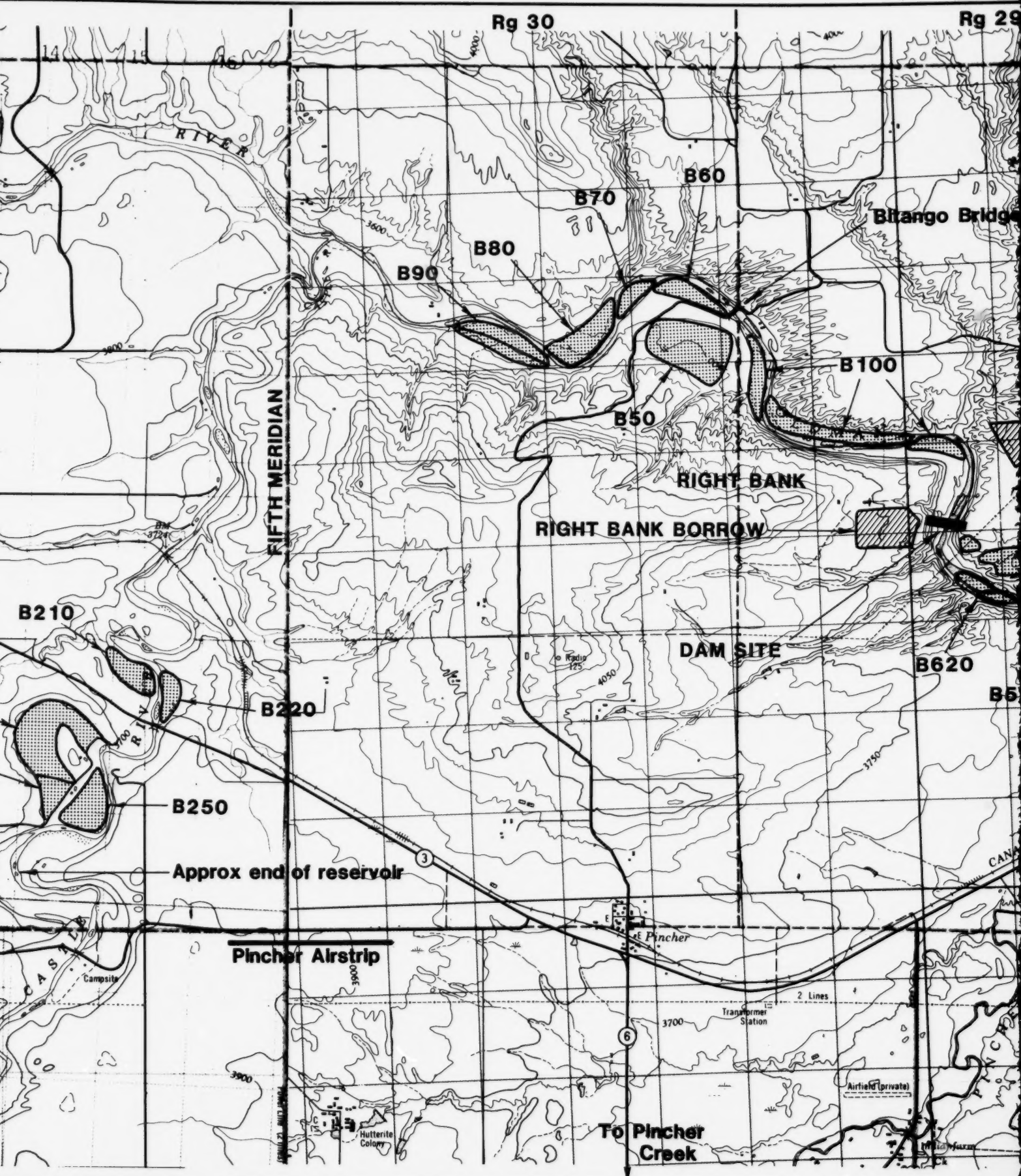
RC ROTARY CORED HOLE

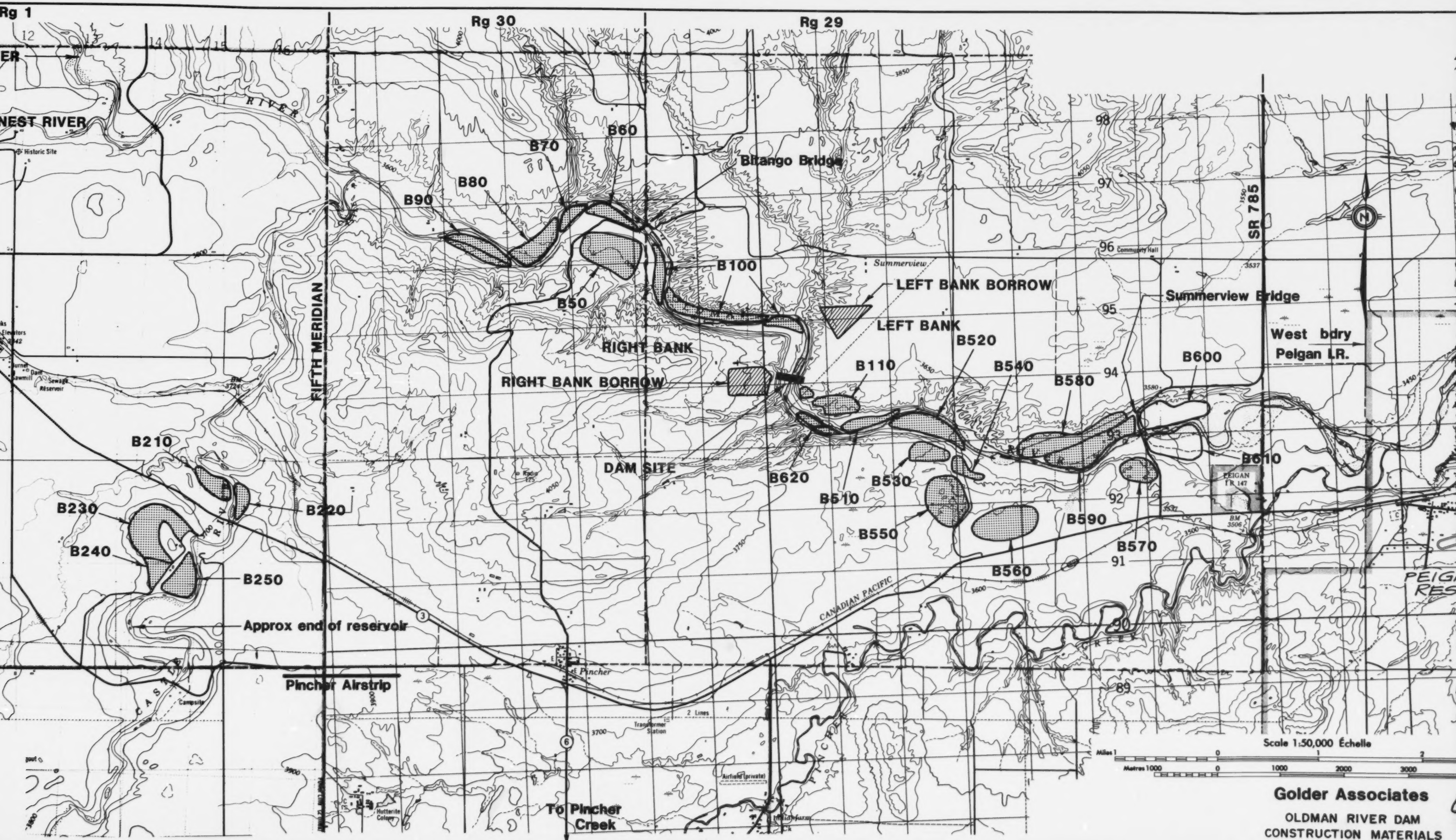
BC BECKER CORED HOLE

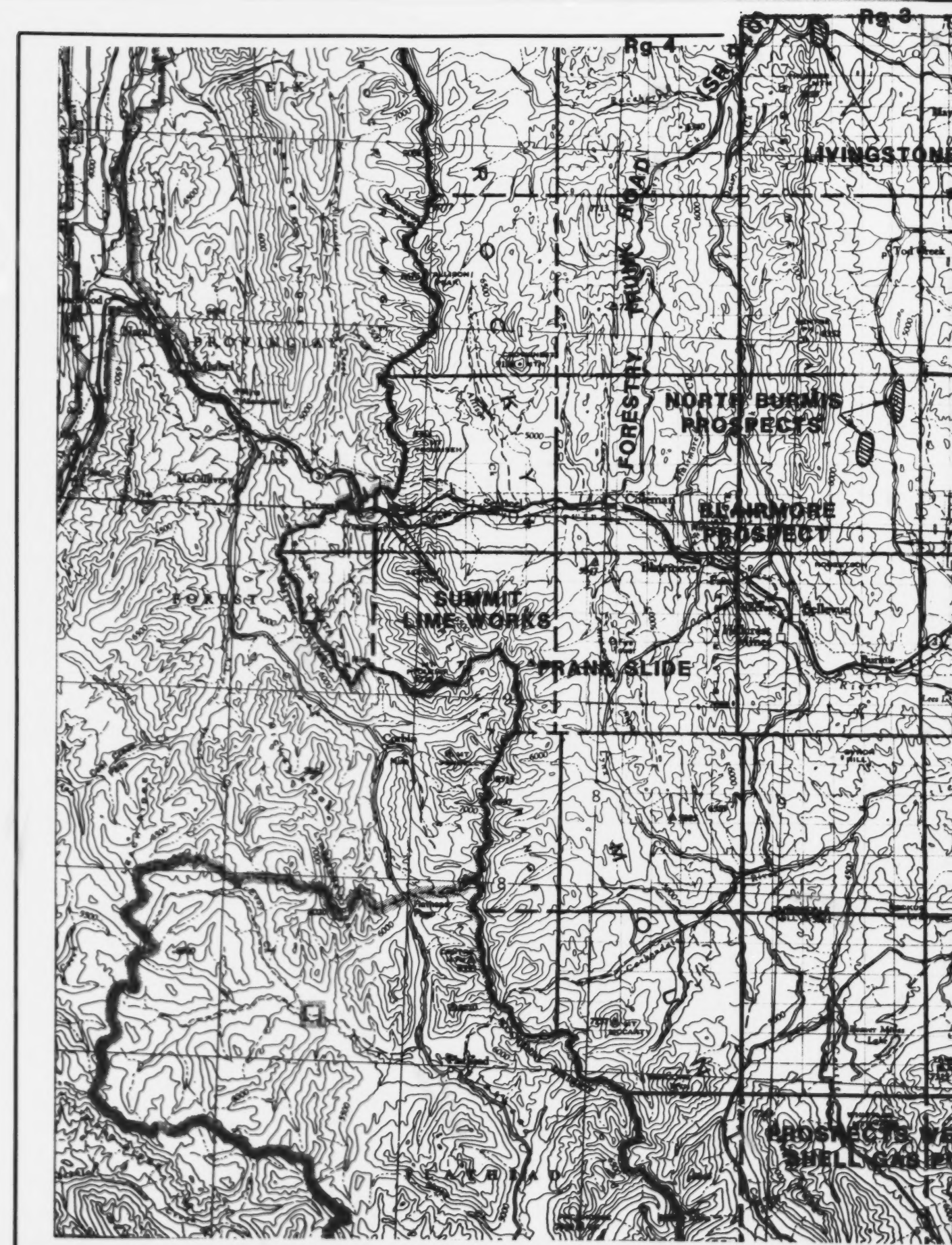
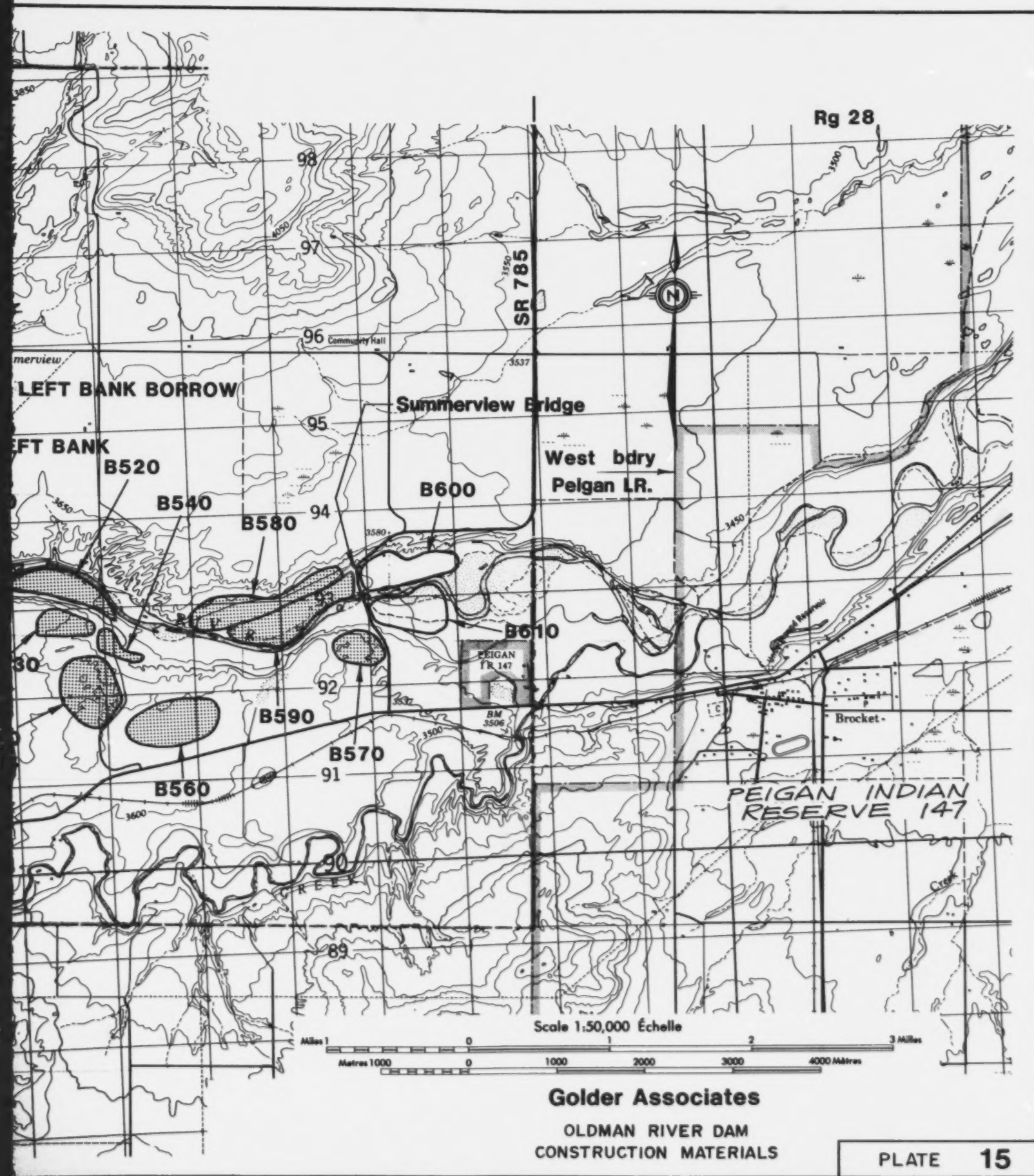
OLDMAN RIVER DAM
GEOLOGICAL SECTION SPILLWAY
PLATE 14

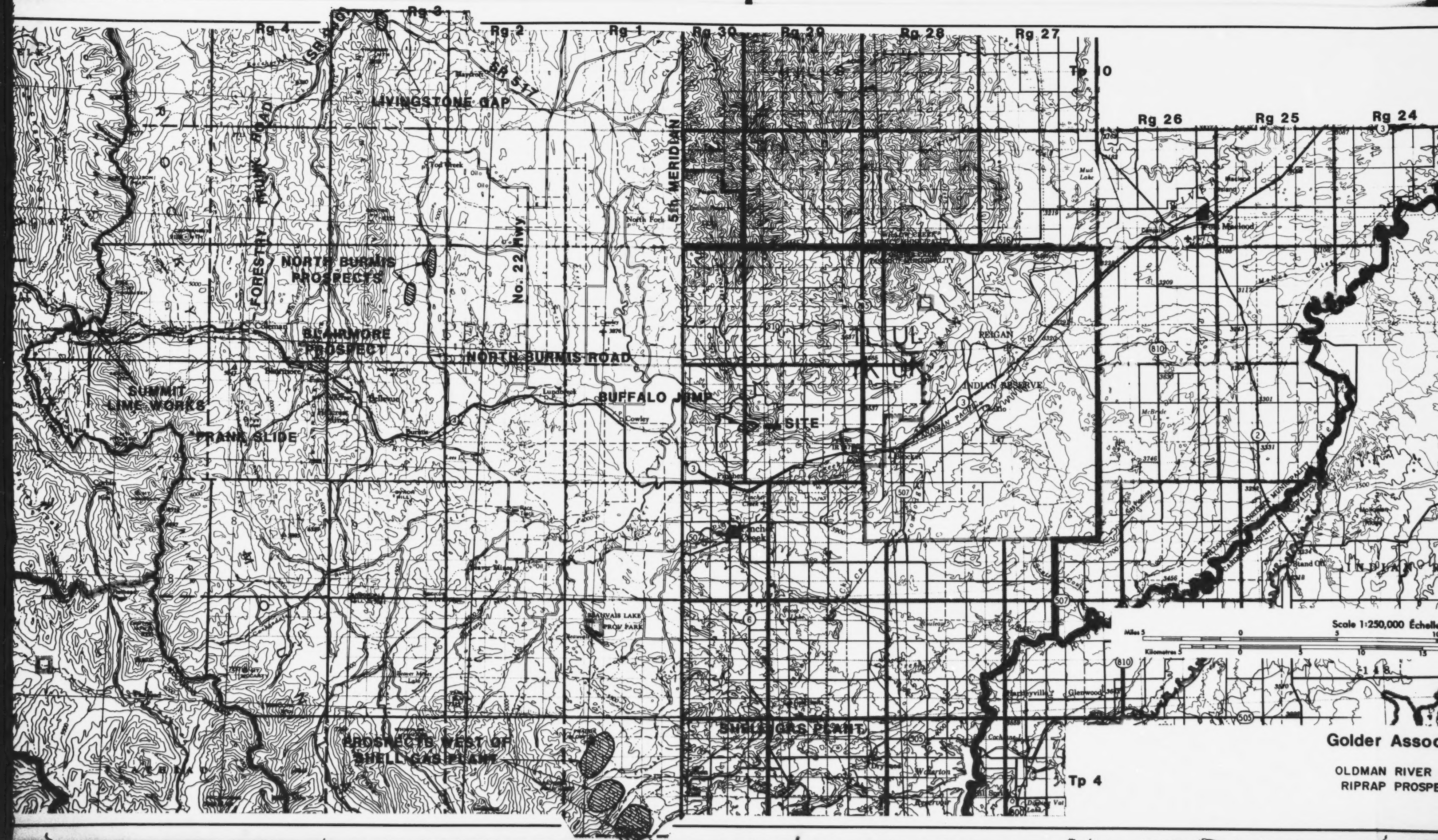




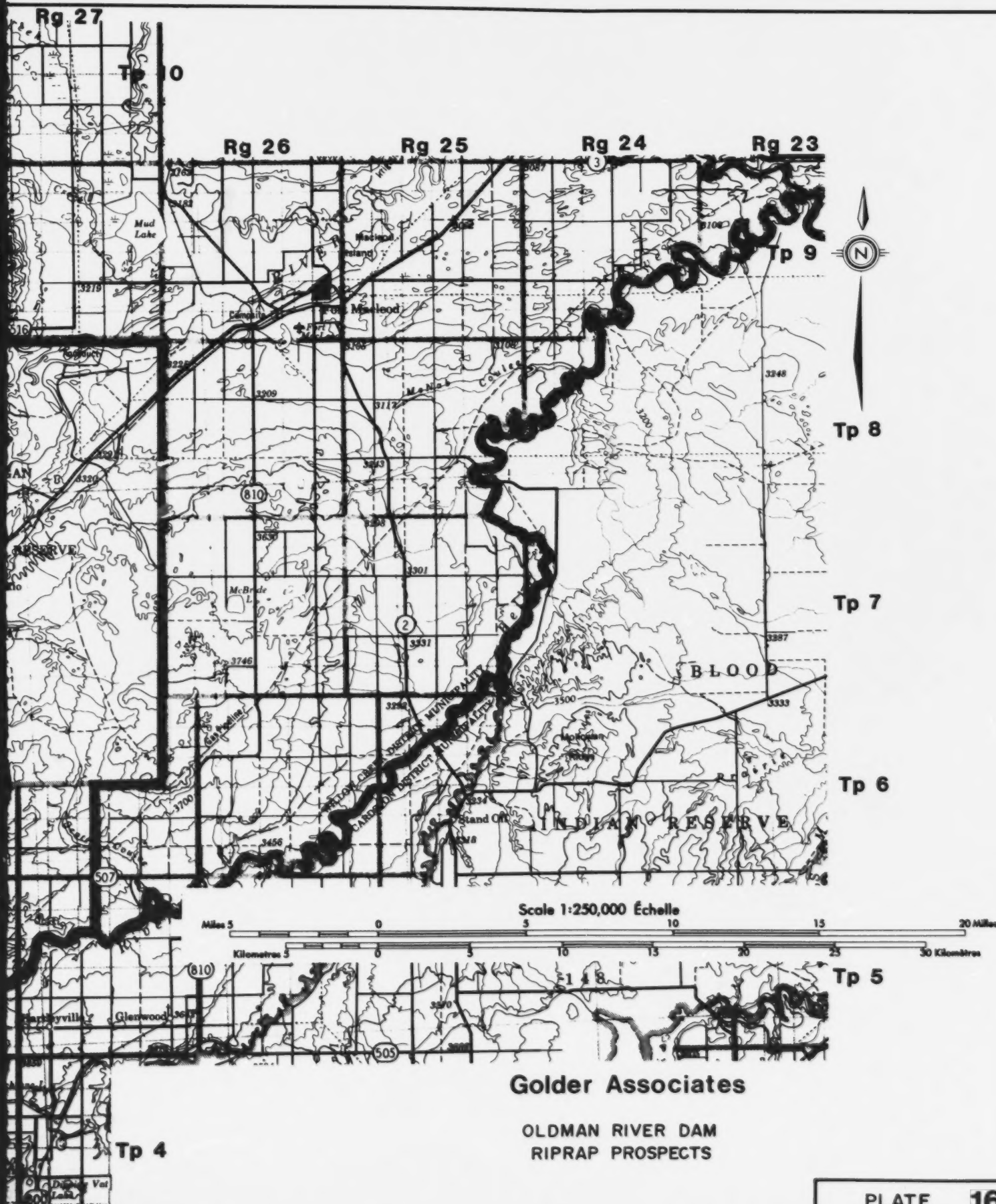


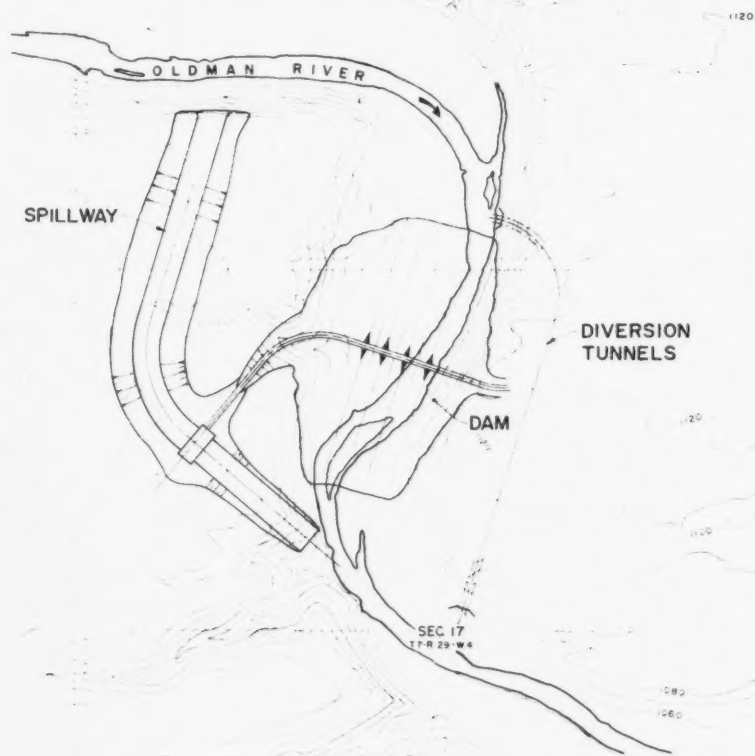




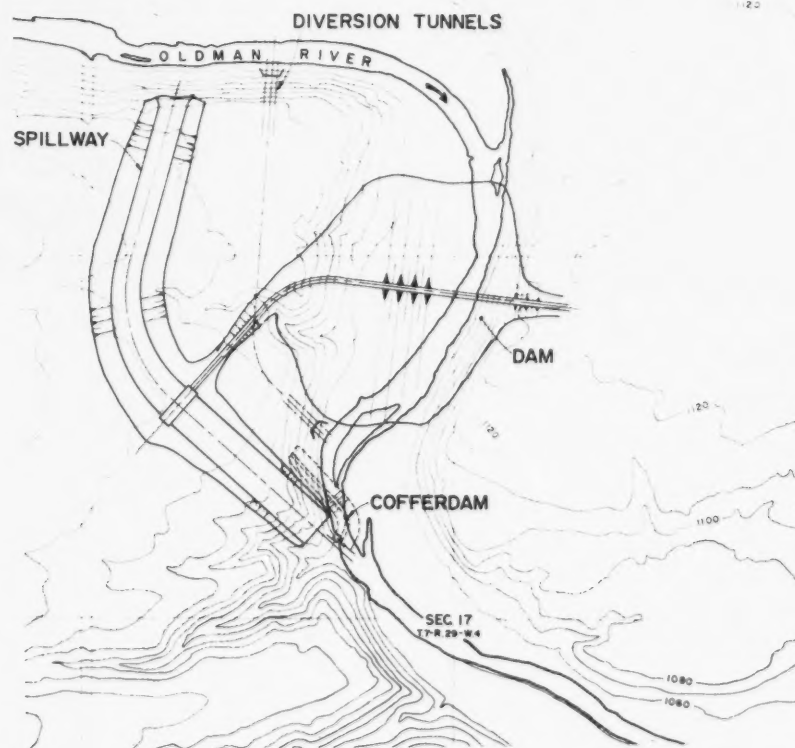
**Golder Assoc**

OLDMAN RIVER
RIPRAP PROSPE

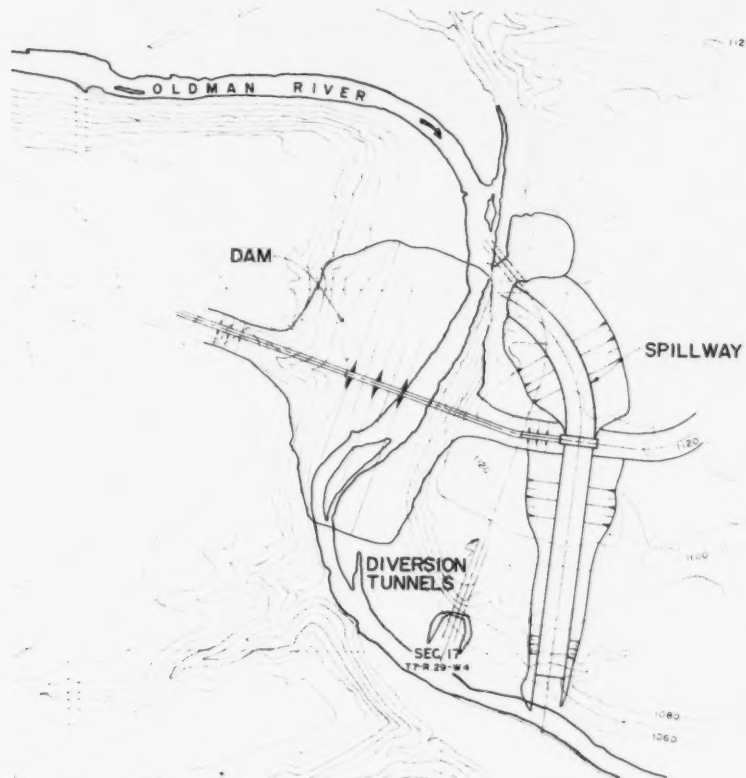




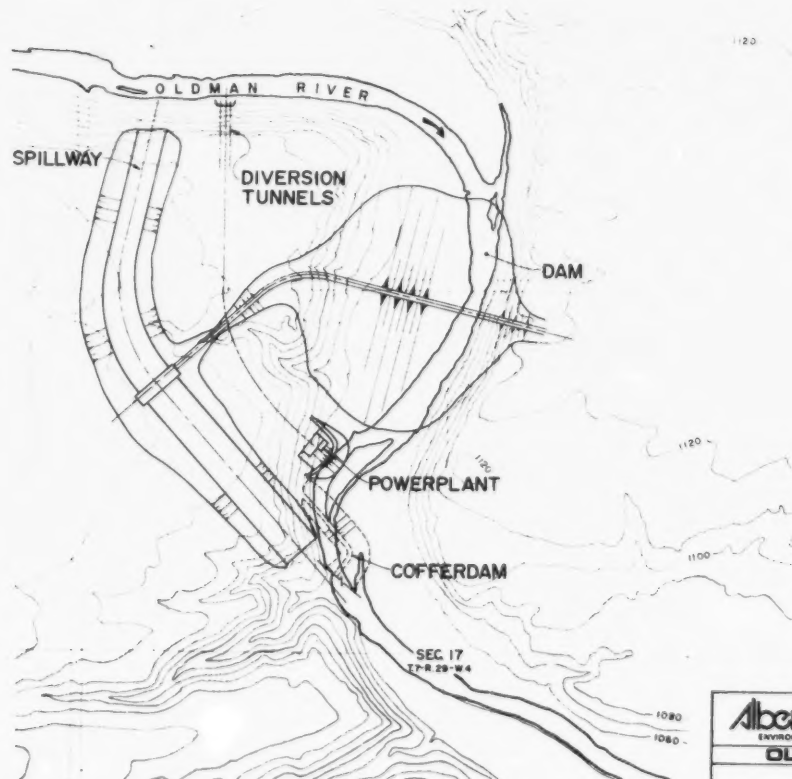
LAYOUT - A



LAYOUT - C

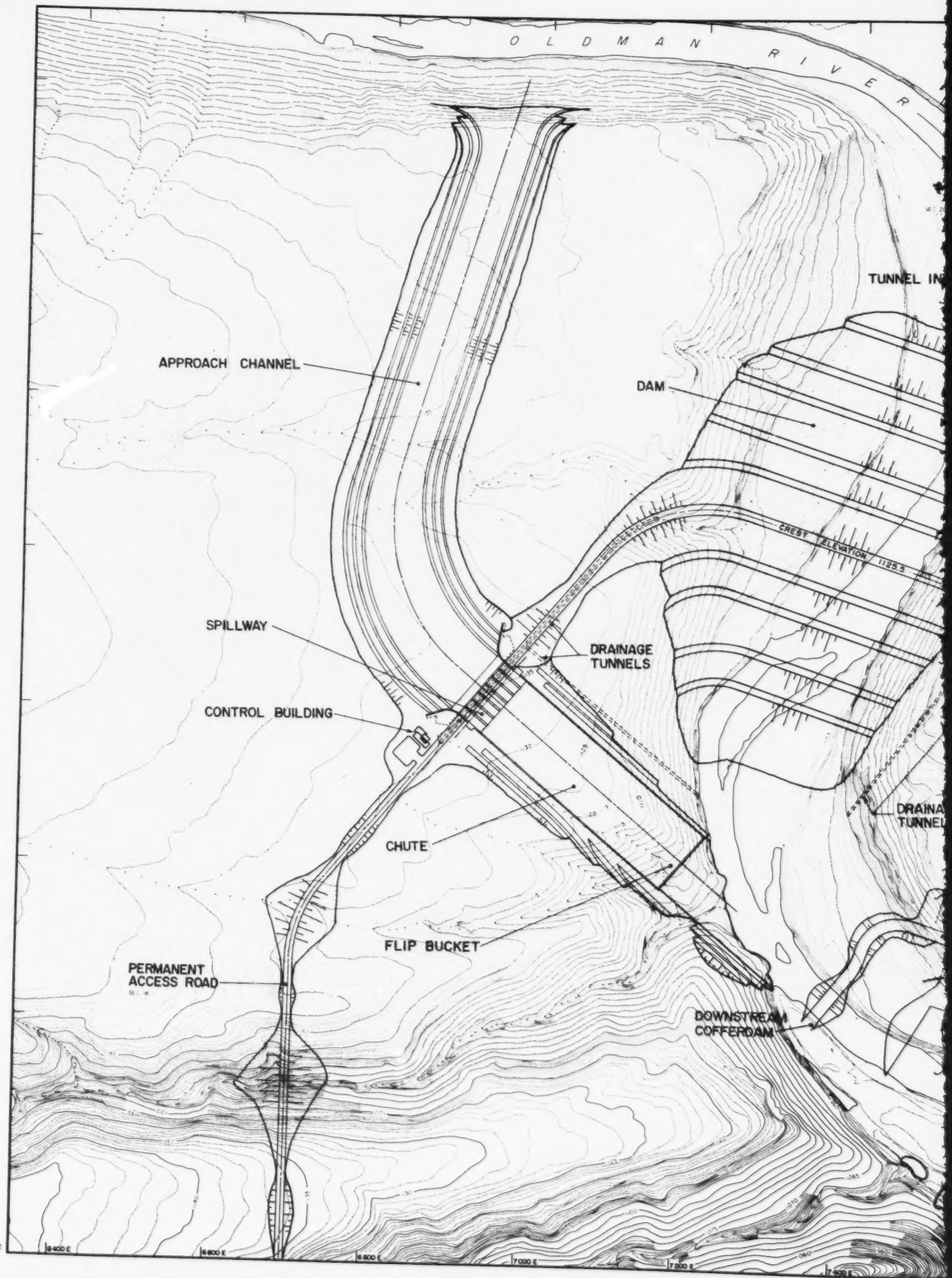


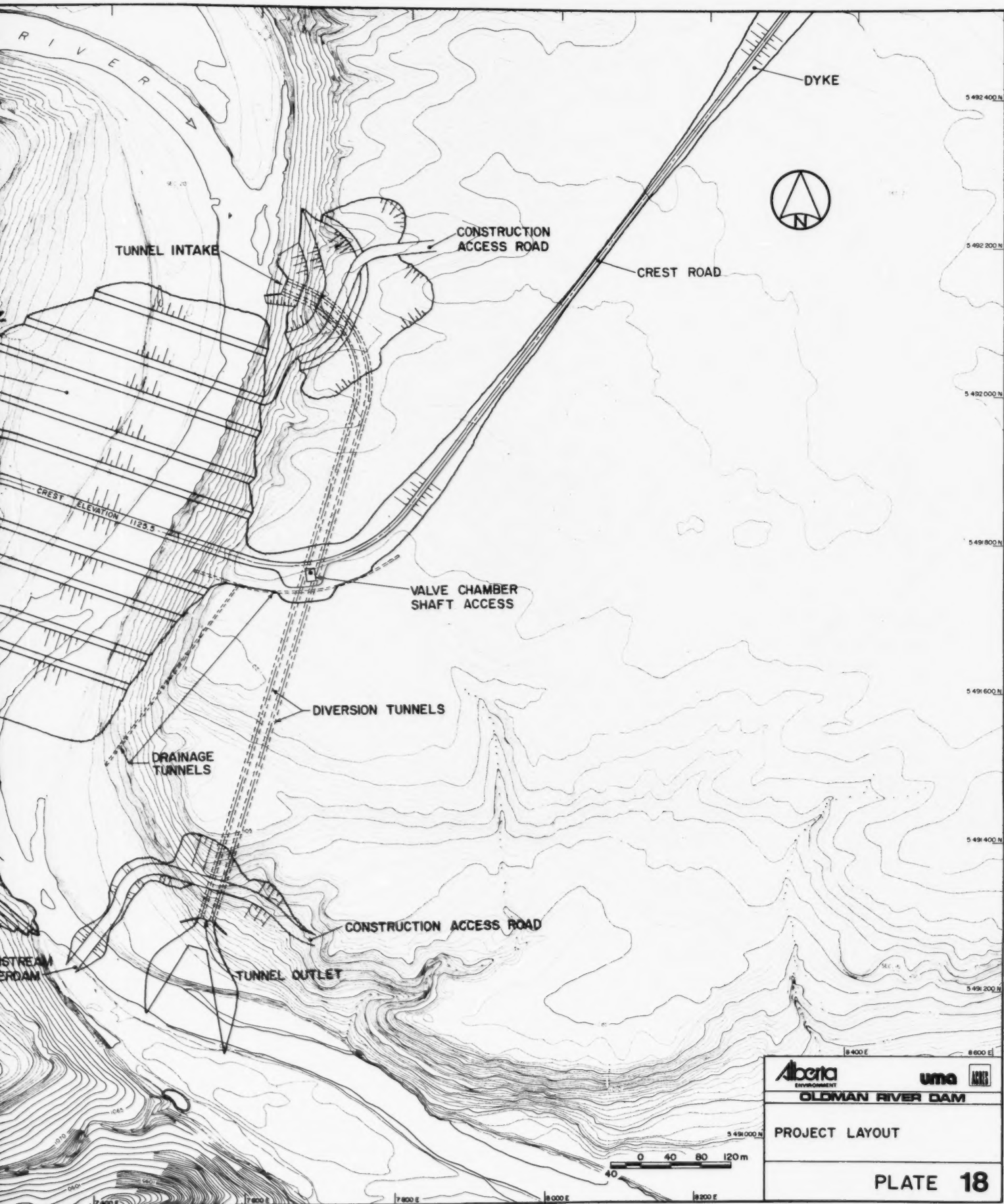
LAYOUT - B

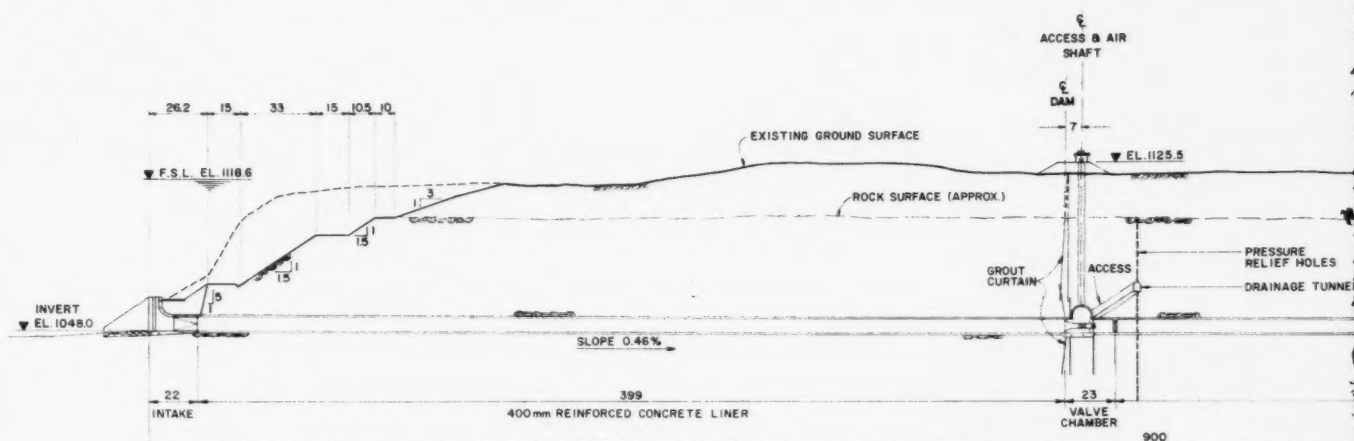


LAYOUT - D

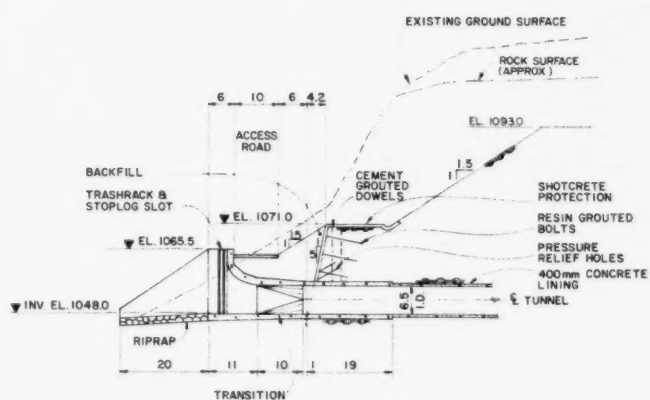
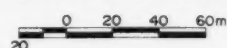
OLDMAN RIVER DAM	
ALTERNATIVE PROJECT LAYOUTS	
PLATE 17	



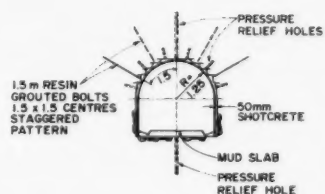
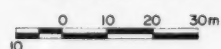




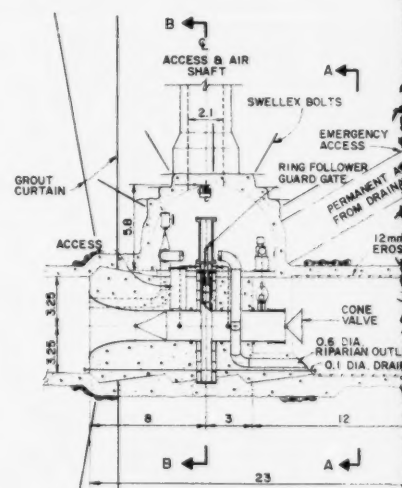
LONGITUDINAL SECT



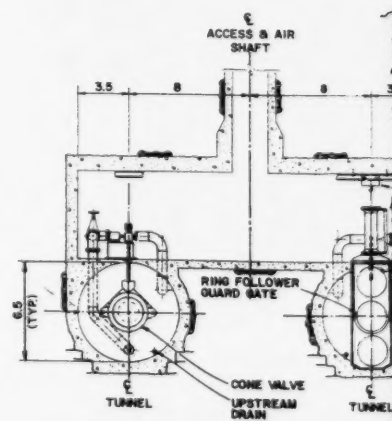
TUNNEL INTAKE



DRAINAGE TUNNEL SECTION

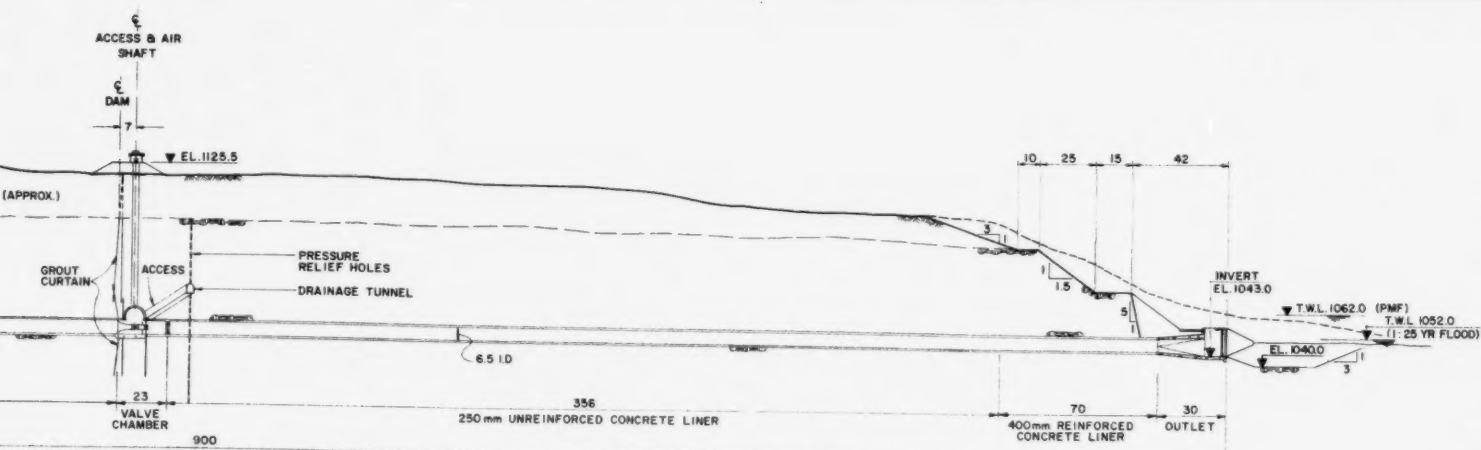


VALVE CHAMBER

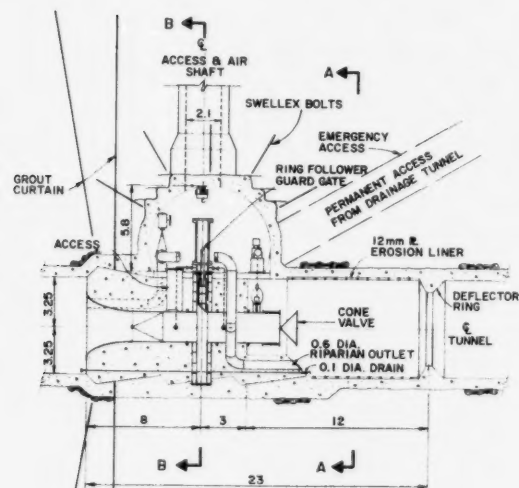
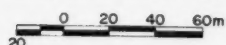


SECTION A-A VALVE CHAMBER

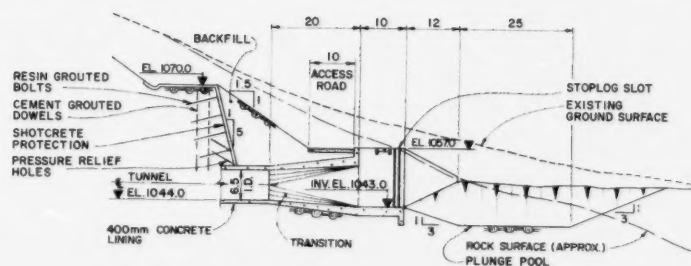
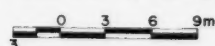




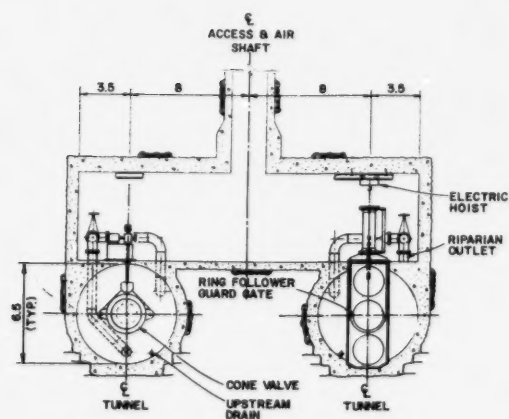
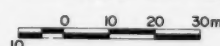
LONGITUDINAL SECTION



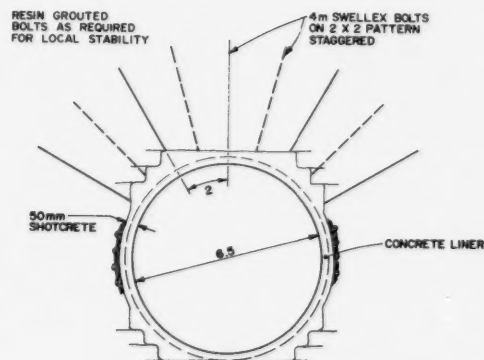
VALVE CHAMBER



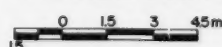
TUNNEL OUTLET



VALVE CHAMBER



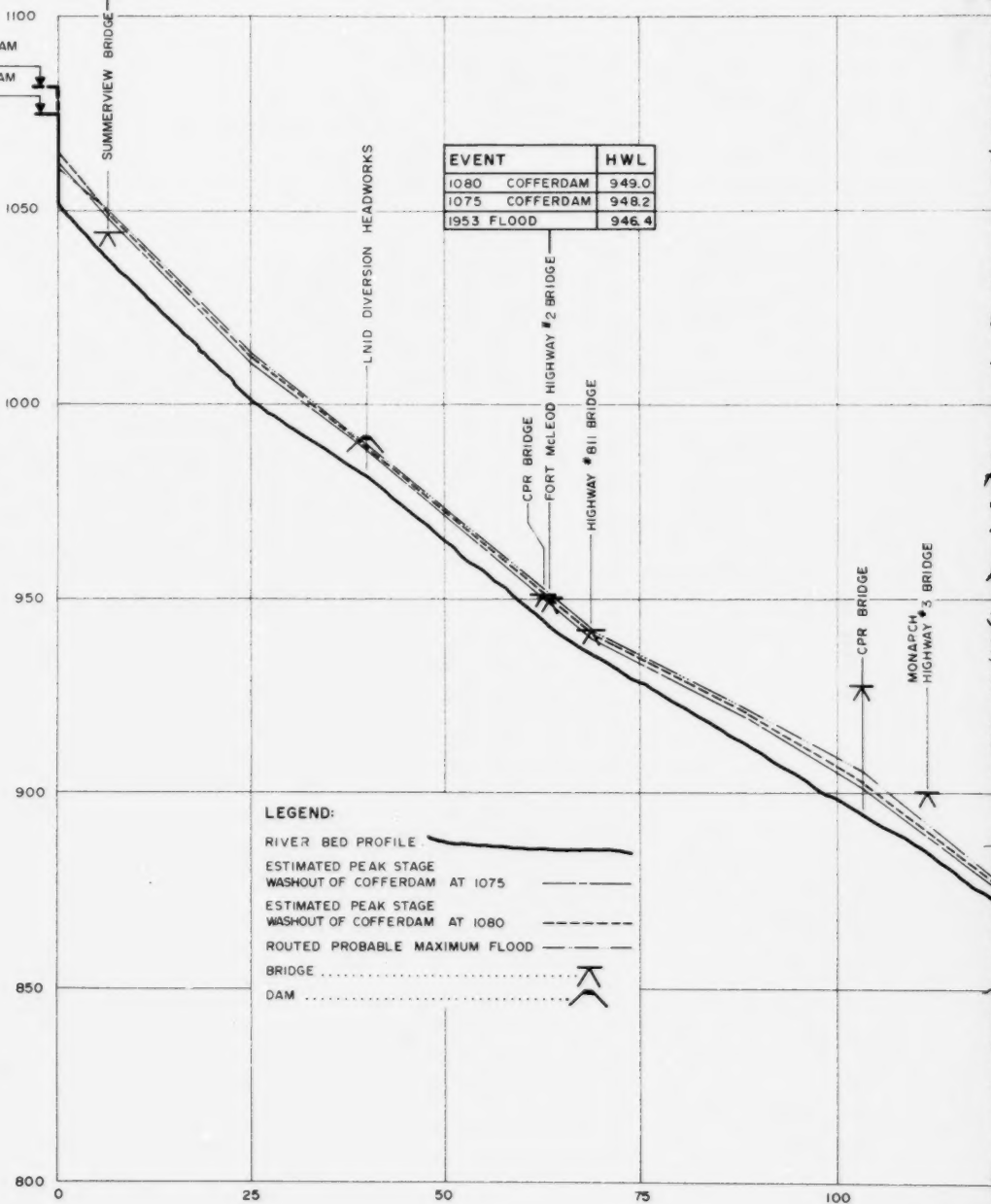
TUNNEL SECTION



ELEVATION (m)

EVENT	HWL
1080 COFFERDAM	1050.0
1075 COFFERDAM	1047.8
1953 FLOOD	1047.2

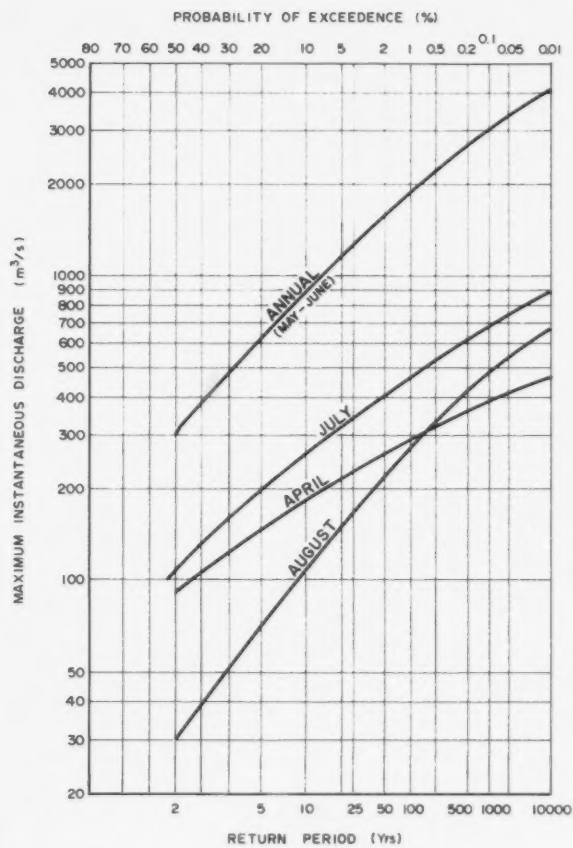
EVENT	HWL
1080 COFFERDAM	949.0
1075 COFFERDAM	948.2
1953 FLOOD	946.4



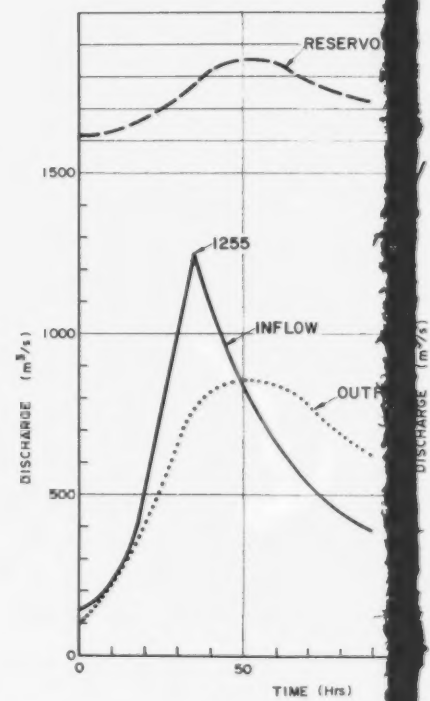
RIVER DISTANCE DOWNSTREAM OF PROJECT S



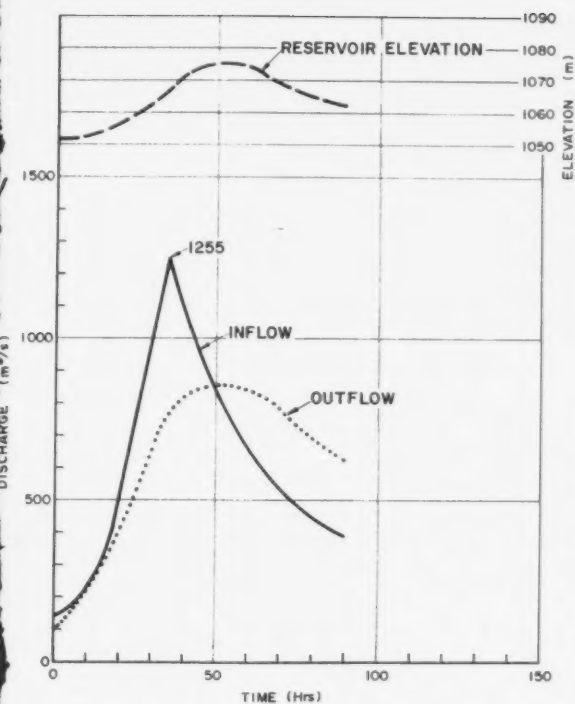
NCE DOWNSTREAM OF PROJECT SITE (km)



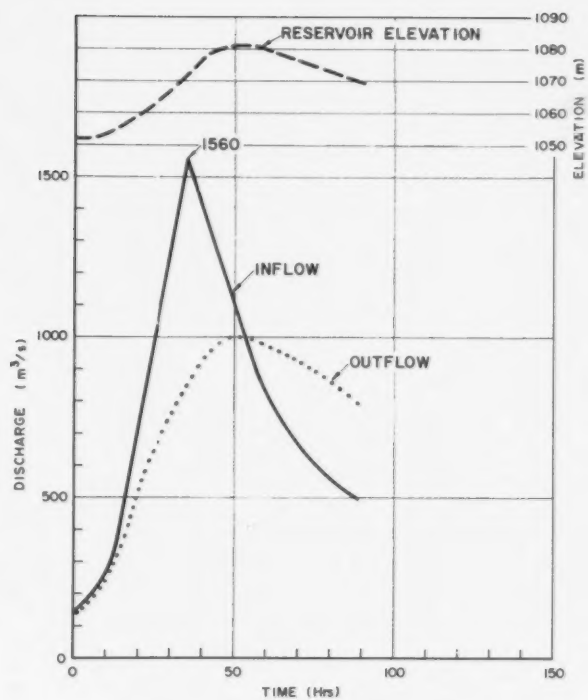
SEASONAL FLOOD FREQUENCY ANALYSIS
OLDMAN RIVER AT PROJECT SITE



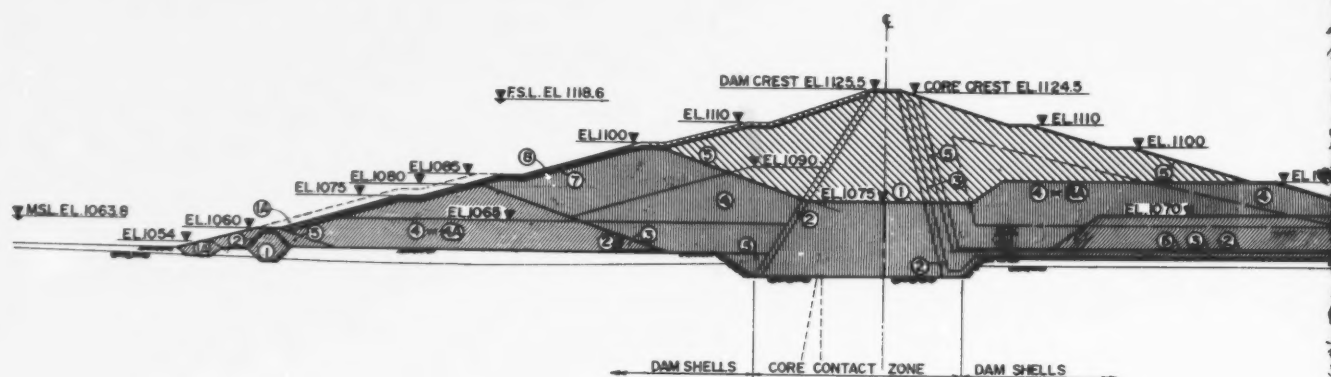
ROUTING OF 1:25 YEAR FLOOD
THROUGH DIVERSION
DURING CONSTRUCTION



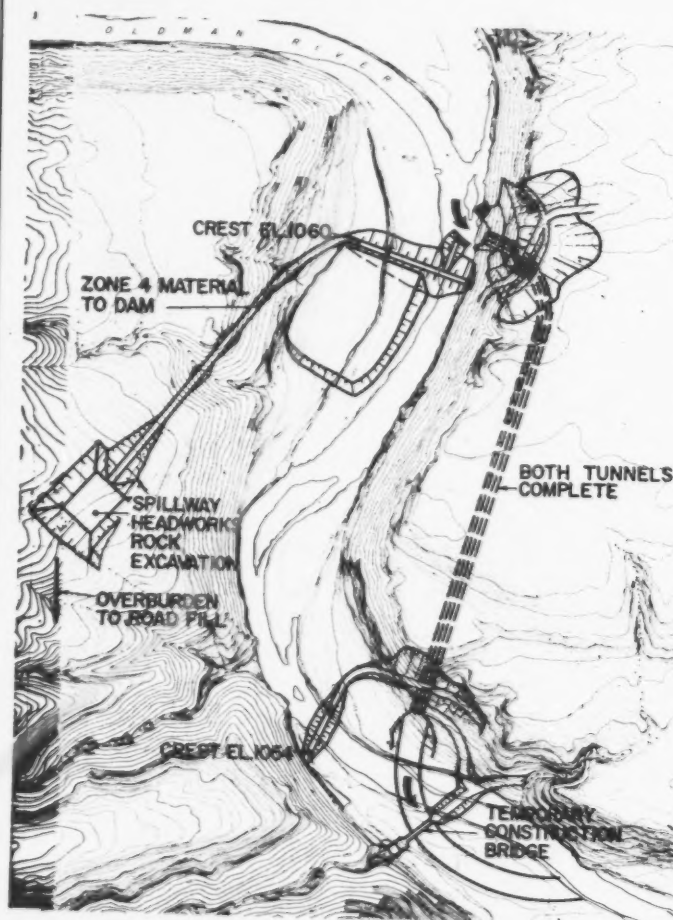
ROUTING OF 1:25 YEAR FLOOD
THROUGH DIVERSION TUNNELS
DURING CONSTRUCTION



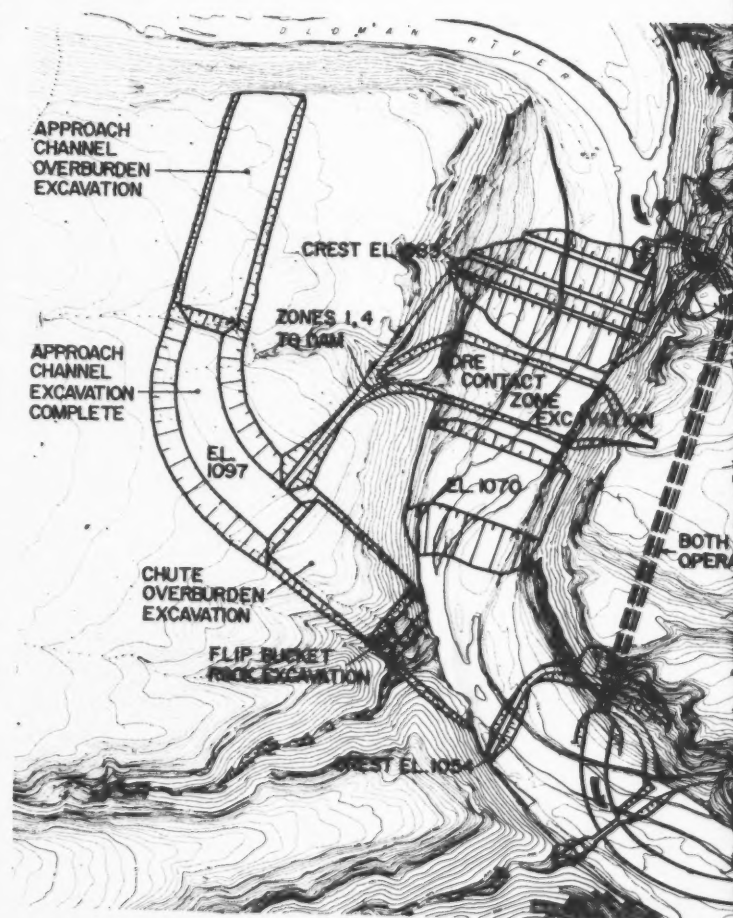
ROUTING OF 1:50 YEAR FLOOD
THROUGH DIVERSION TUNNELS
DURING CONSTRUCTION



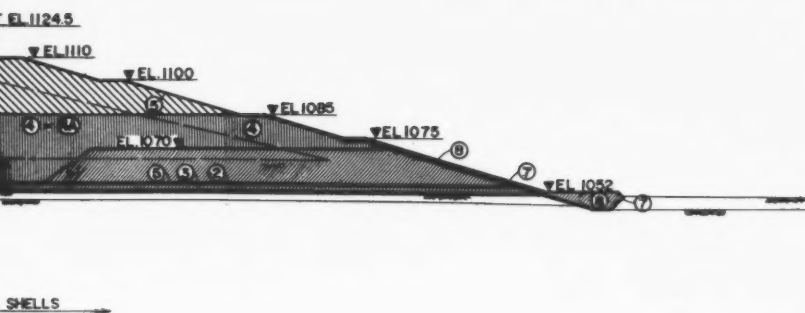
DAM SECTION



JULY 1988



APRIL 1989

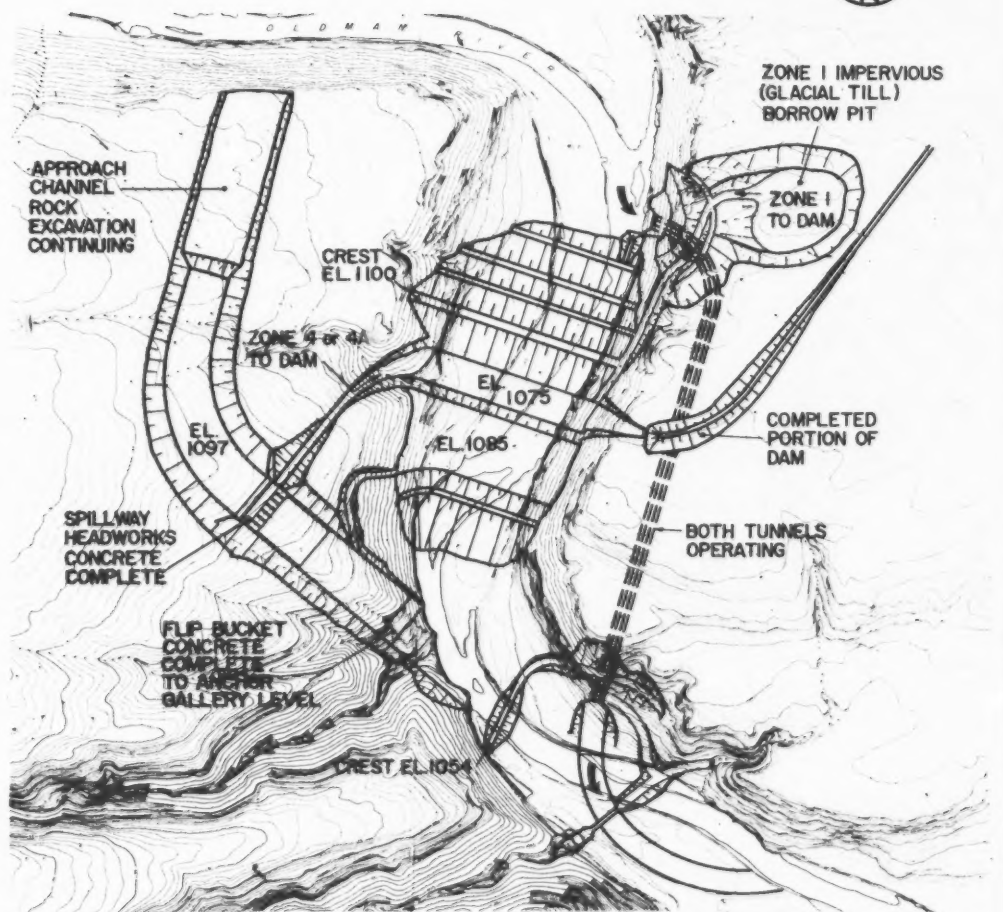


LEGEND

- COMPLETED 1988 SEASON
- COMPLETED 1989 SEASON
- COMPLETED 1990 SEASON

ZONE

- ① IMPERVIOUS - GLACIAL TILL
- ①A IMPERVIOUS - RANDOM
- ② TRANSITION
- ③ DRAIN
- ④ RANDOM FILL - ACCEPTABLE ROCK
- ④A RANDOM FILL - ACCEPTABLE SOIL
- ⑤ SELECTED PIT RUN SAND AND GRAVEL
- ⑥ NOT USED
- ⑦ RIPRAP BEDDING
- ⑧ RIPRAP



1989

NOVEMBER 1989

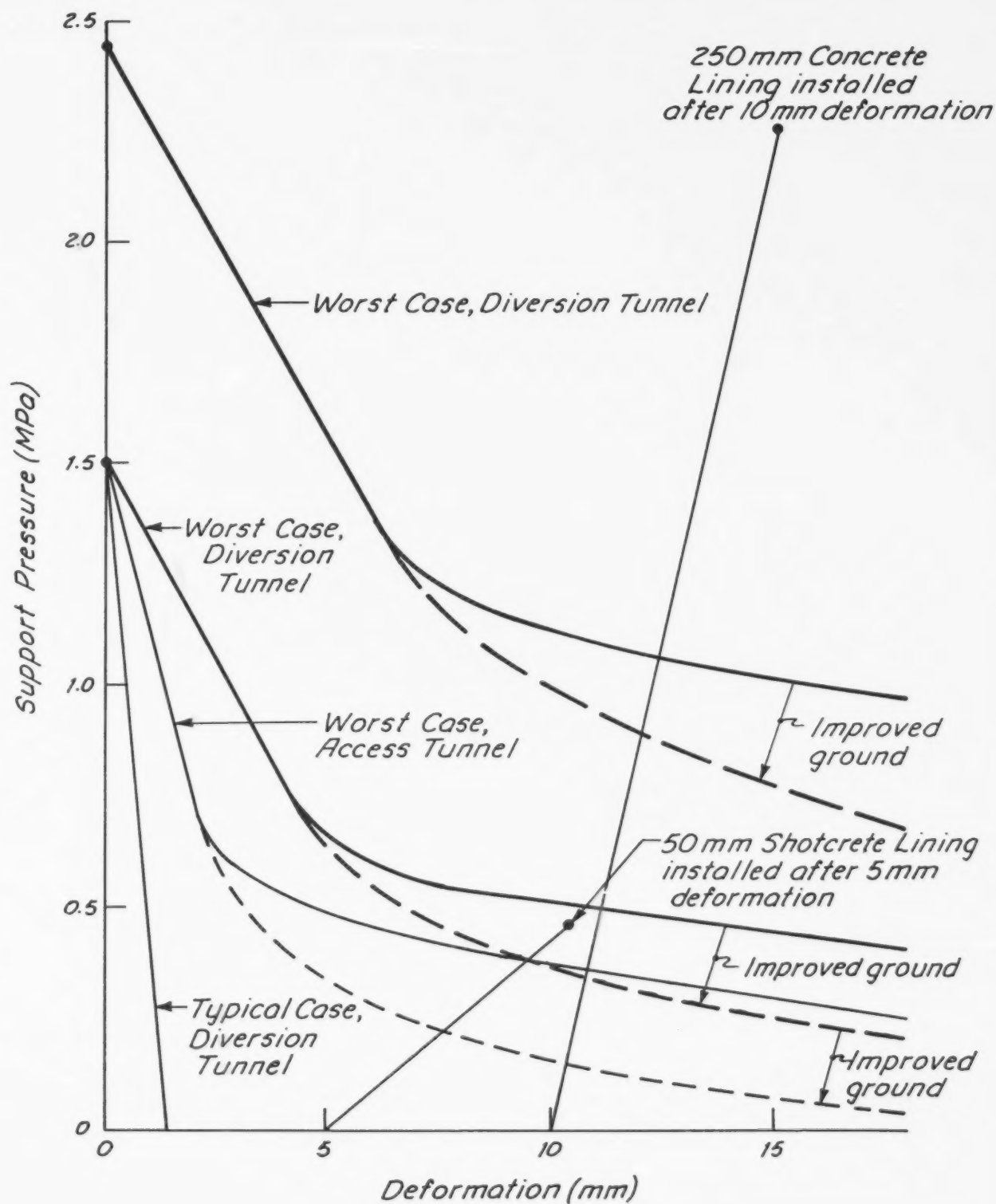


OLDMAN RIVER DAM

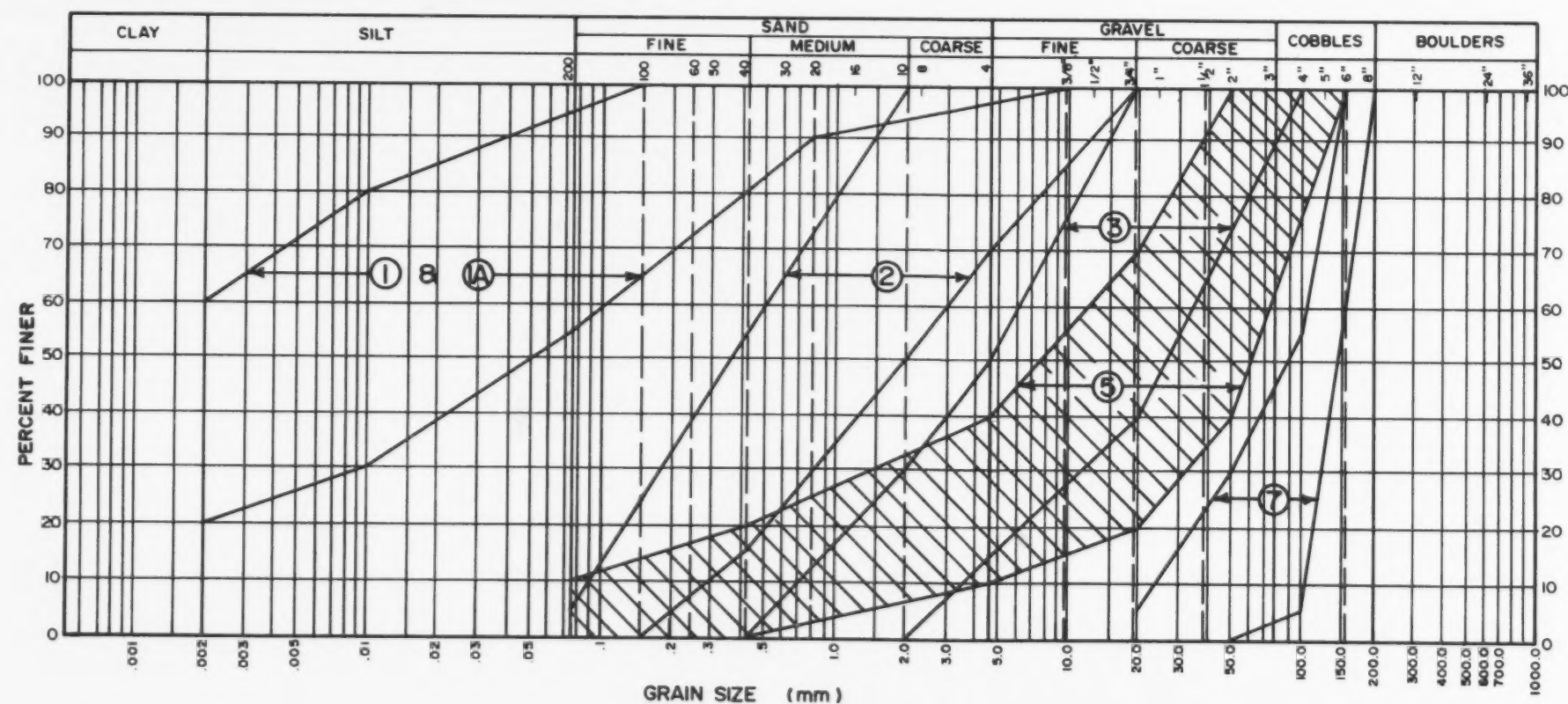
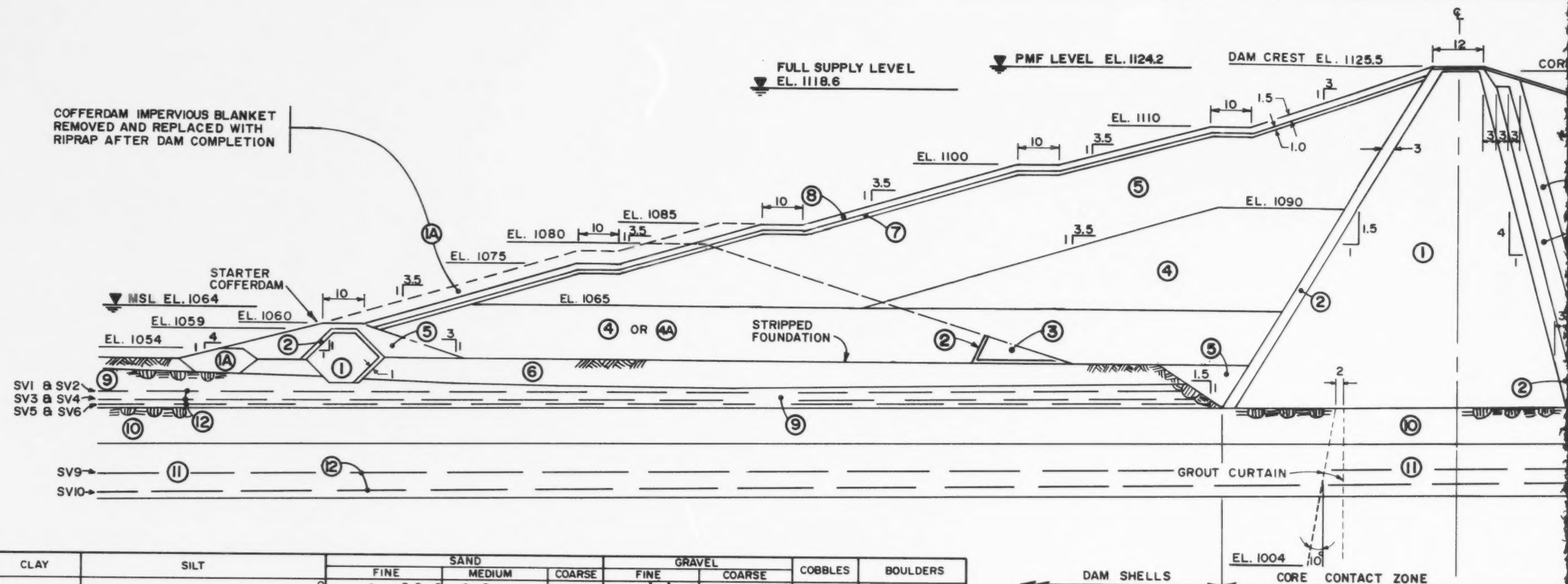
CONSTRUCTION SEQUENCE

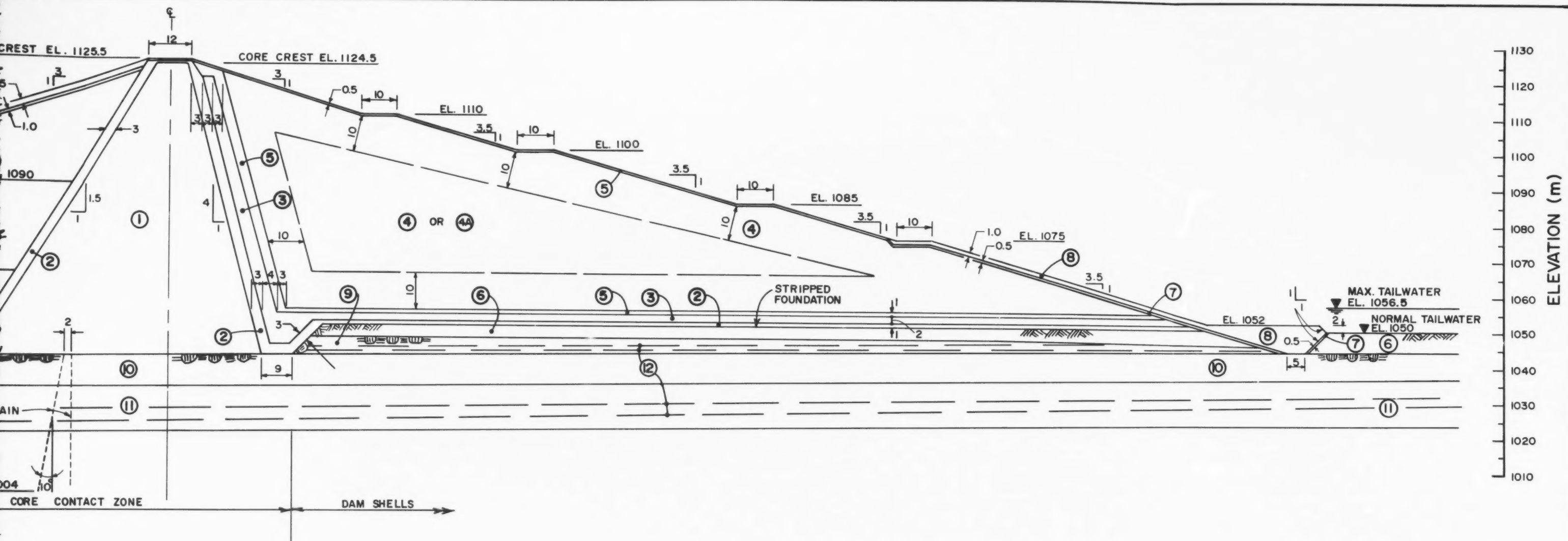
PLATE 22

ROCK SUPPORT INTERACTION ANALYSIS PRELIMINARY DESIGN



ELEVATION (m)



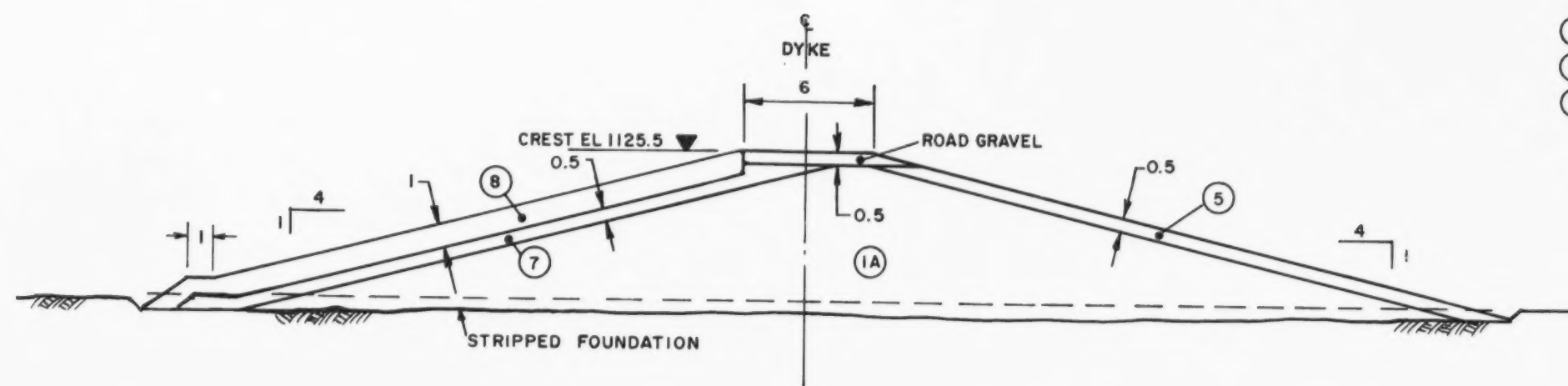


LEGEND:

- ① IMPERVIOUS - (GLACIAL TILL)
- ④A IMPERVIOUS - (RANDOM)
- ② FILTER
- ③ DRAIN
- ④ RANDOM FILL - ACCEPTABLE ROCK FROM REQUIRED EXCAVATION
- ④A RANDOM FILL - ACCEPTABLE SOIL FROM REQUIRED EXCAVATION
- ⑤ SELECTED PIT RUN SAND AND GRAVEL
- ⑥ EXISTING ALLUVIUM AND COLLUVIUM
- ⑦ RIPRAP BEDDING
- ⑧ RIPRAP
- ⑨ CLAYSTONE (UNITS J10 AND J11)
- ⑩ SANDSTONE (UNITS K1 AND K2)
- ⑪ CLAYSTONE / SANDSTONE INTERBEDS (UNIT L)
- ⑫ SEAMS SVI-SV10

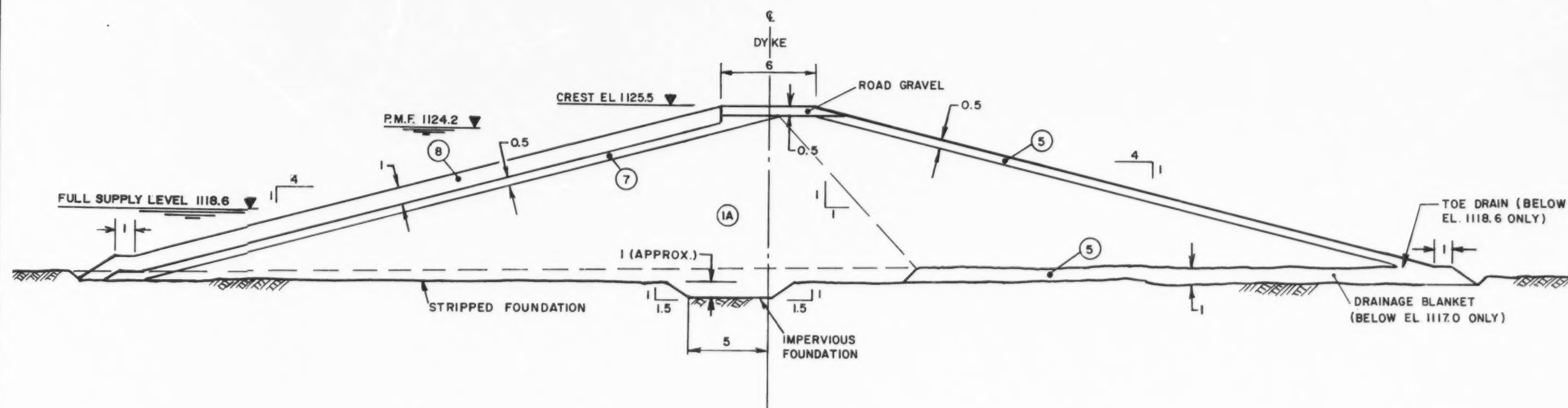
LEGEND:

- (1A) IMPERVIOUS (RANDOM) INCLUDING GLACIOLACUSTRINE SILT & CLAY
- (5) SELECTED PITRUN SAND & CLAY
- (7) RIPRAP BEDDING
- (8) RIPRAP



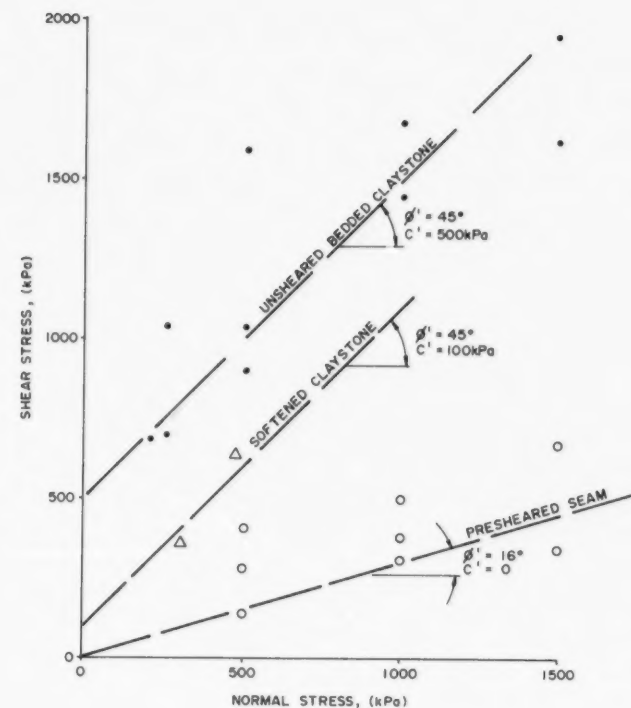
FREEBOARD DYKE SECTION

(FOUNDATION ABOVE EL. 1118.6)



TYPICAL DYKE SECTION



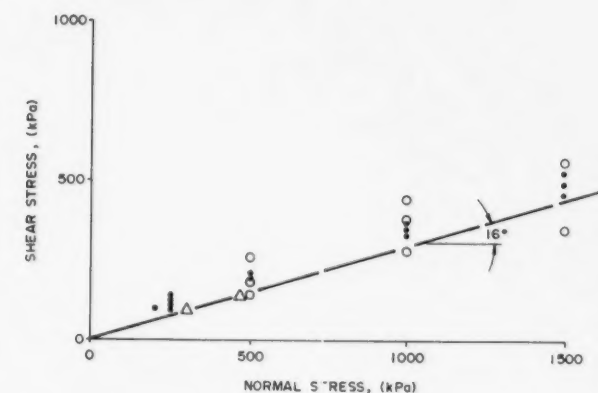


PEAK ANGLE OF SHEARING RESISTANCE

INDEX PROPERTIES (RANGE)			
	CLAYSTONE	SEAMS	SOFTENED CLAYSTONE
MC	5.7 - 6.4	10.0 - 17.8	14.0
LL	35.4 - 36.7	32.4 - 35.6	46.6
PL	18.8 - 20.2	18.9 - 22.2	22.9
PI	15.6 - 16.7	10.5 - 15.9	23.7
CLAY %	32.0 - 47.5	36.3 - 41.8	46.0
SILT %	51.5 - 67.0	57.2 - 61.0	54.0
SAND %	1	1.0 - 2.7	0
ACTIVITY	0.35 - 0.49	0.29 - 0.40	0.52

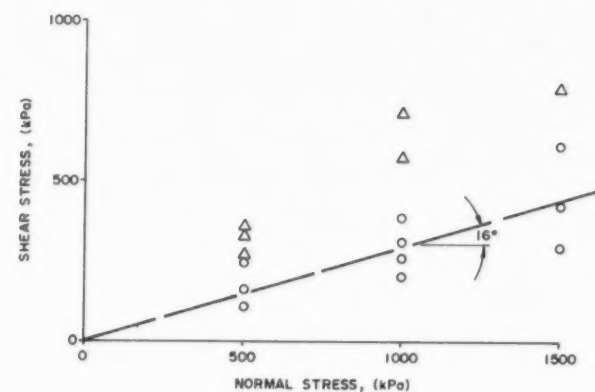
LEGEND

- UNSHEARED BEDDED CLAYSTONE
- PRESHEARED SEAM
- △ SOFTENED CLAYSTONE



RESIDUAL ANGLE OF SHEARING RESISTANCE

DIRECT SHEAR TEST RESULTS
CLAYSTONE UPPER MUDROCK SEQUENCE UNITS J10 AND J11

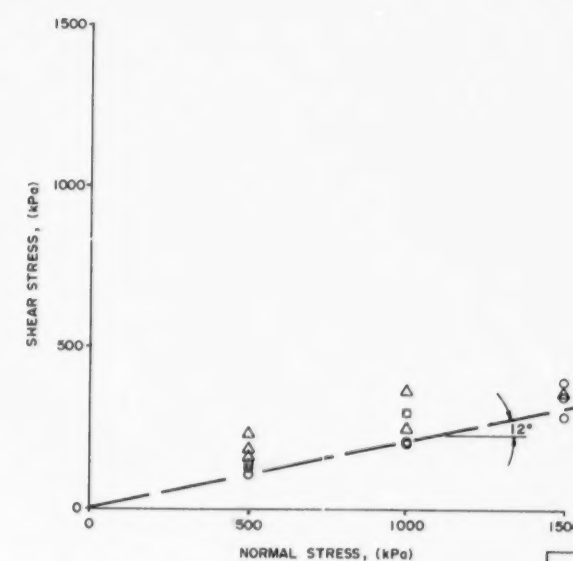


PEAK ANGLE OF SHEARING RESISTANCE

INDEX PROPERTIES (RANGE)		
	SEAMS FROM SHAFT 100 x 100mm SAMPLES	SEAMS FROM DRILLHOLES 50 x 50 mm SAMPLES
MC	9.0 - 13.9	14.3 - 17.9
LL	33.9 - 42.1	34.0 - 38.3
PL	20.6 - 24.0	16.2 - 18.1
PI	13.0 - 20.0	17.8 - 20.2
CLAY %	33.0 - 55.3	35.0 - 42.0
SILT %	44.7 - 66.0	49.0 - 55.0
SAND %	0.0 - 2.0	9.0 - 10.0
ACTIVITY	0.34 - 0.44	0.42 - 0.58

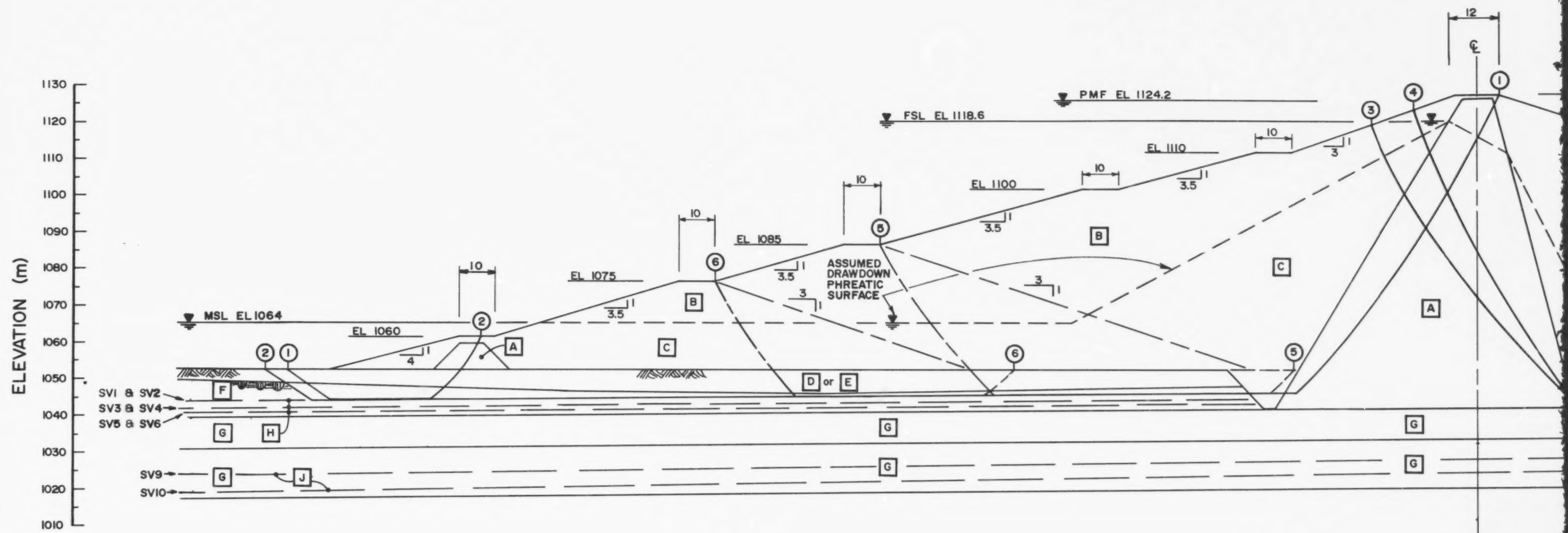
LEGEND

- SEAMS FROM SHAFT
- △ SEAMS FROM DRILL HOLES

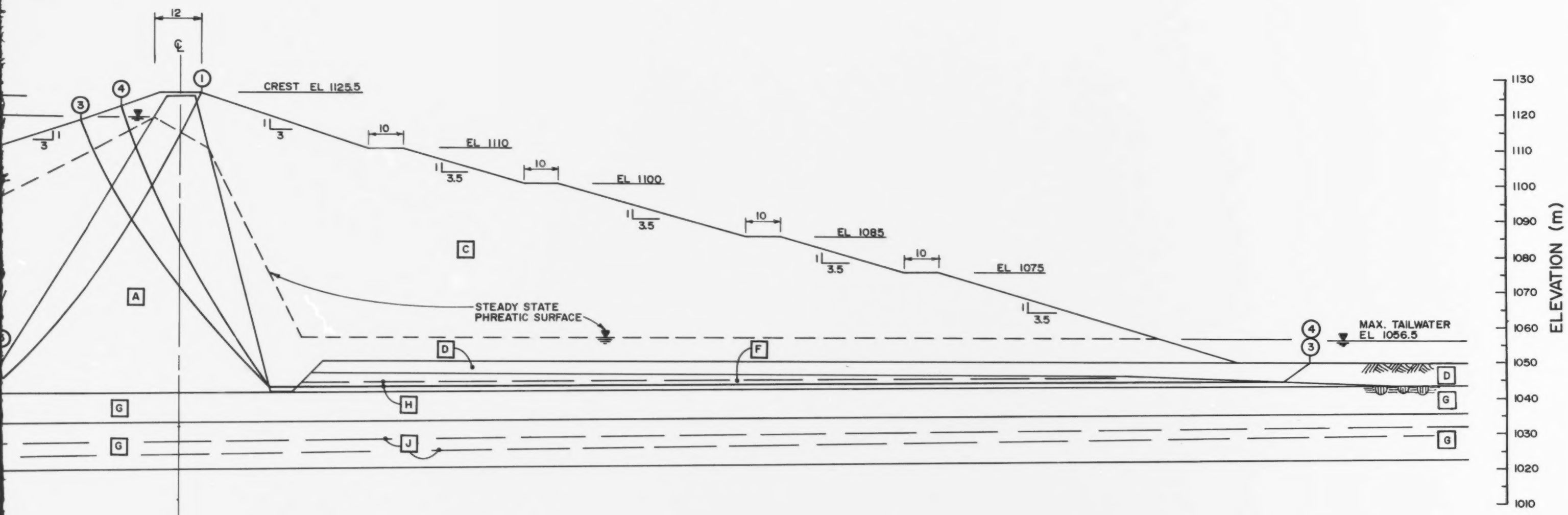


RESIDUAL ANGLE OF SHEARING RESISTANCE

DIRECT SHEAR TEST RESULTS
CLAYSTONE STRUCTURAL UNITS #1 AND #2 - UNIT L

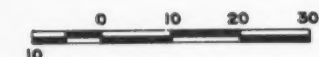


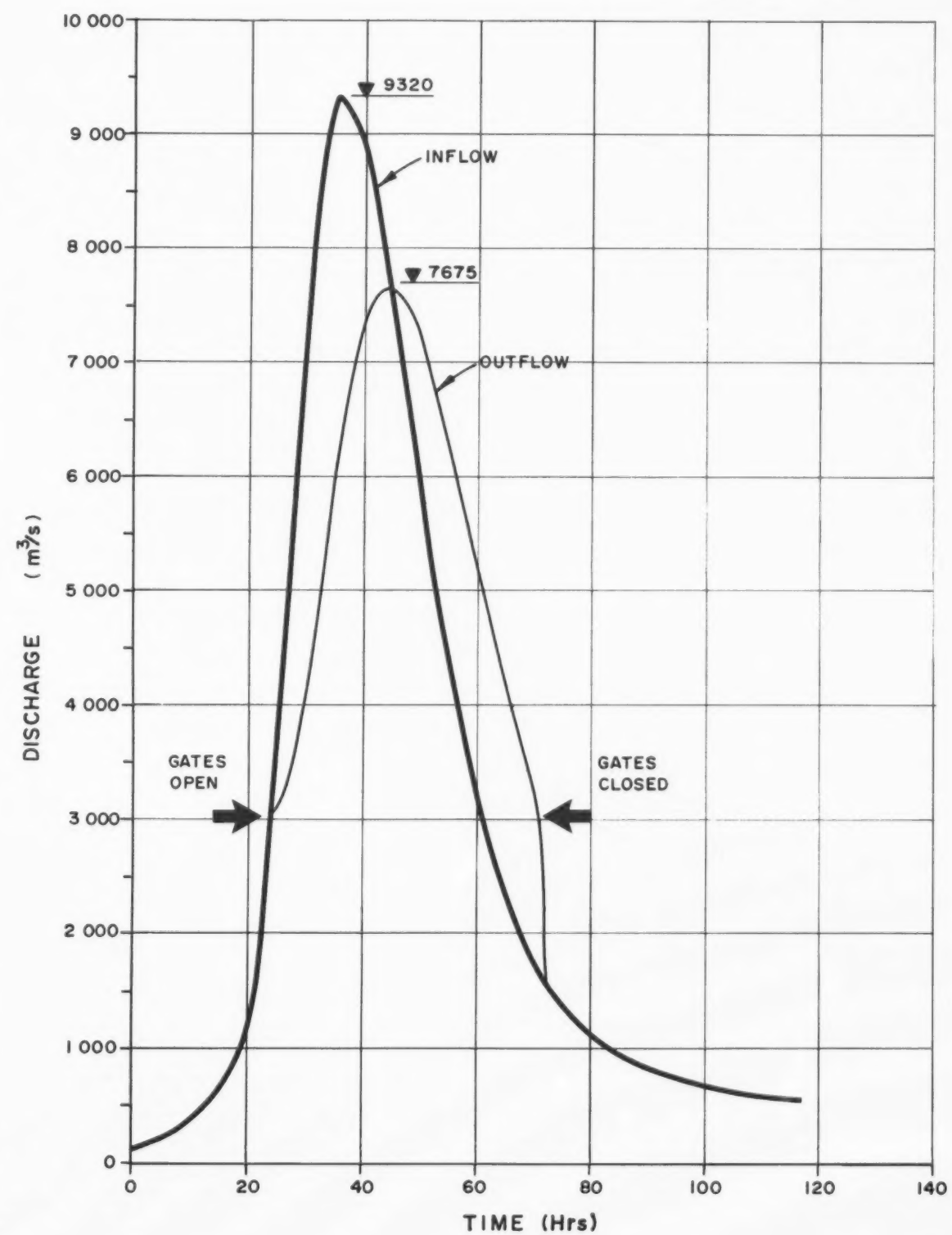
CASE	FACTOR OF SAFETY F. S.	MIN. F. S. REQ'D.	UPSTREAM		DOWNSTREAM		CASE	FACTOR OF SAFETY F. S.	MIN. F. S. REQ'D.	UPSTREAM		DOWNSTREAM		CASE	FACTOR OF SAFETY F. S.	MIN. F. S. REQ'D.	UPSTREAM		DOWNSTREAM		CASE
			F. S.	FAILURE SURFACE	F. S.	FAILURE SURFACE				F. S.	FAILURE SURFACE	F. S.	FAILURE SURFACE				F. S.	FAILURE SURFACE	F. S.	FAILURE SURFACE	
1. RESERVOIR AT FULL SUPPLY LEVEL WITH STEADY STATE SEEPAGE CONDITIONS IN THE DAM AND FOUNDATION.		1.5	1.9	1	1.5	3	4. RAPID RESERVOIR DRAWDOWN FROM FULL SUPPLY LEVEL TO MINIMUM SUPPLY LEVEL.		1.3	1.3	1	-	-	7. RESERVOIR AT PROBABLE MAXIMUM FLOOD LEVEL WITH STEADY STATE SEEPAGE CONDITIONS IN THE DAM AND FOUNDATION.		1.2	1.9	1	1.4	3	10. COFFERDAM CREST ELEVATION 1,085 m. RIVER LEVEL ELEVATION 1,083. WITH STEADY STATE SEEPAGE CONDITIONS IN COFFERDAM AND FOUNDATION.
2. RESERVOIR AT FULL SUPPLY LEVEL WITH STEADY STATE SEEPAGE CONDITIONS IN THE DAM AND FOUNDATION PLUS PSEUDO-STATIC SEISMIC HORIZONTAL LOADING 0.065g.		1.0	1.2	1	1.1	3	5. RAPID RESERVOIR DRAWDOWN FROM FULL SUPPLY LEVEL TO MINIMUM SUPPLY LEVEL PLUS PSEUDO-STATIC SEISMIC HORIZONTAL LOADING 0.065g.		1.0	1.0	1	-	-	8. COFFERDAM CREST ELEVATION 1,085 m. RIVER LEVEL ELEVATION 1,083. WITH STEADY STATE SEEPAGE CONDITIONS IN COFFERDAM AND FOUNDATION.		1.4	2.1	2	1.6	5	11. COFFERDAM CREST ELEVATION 1,085. END OF CONSTRUCTION PORE PRESSURES IN COFFERDAM AND FOUNDATION.
3. RESERVOIR AT FULL SUPPLY LEVEL WITH STEADY STATE SEEPAGE CONDITIONS IN DAM AND CONSTRUCTION PORE PRESSURES IN FOUNDATION.		1.5	-	-	$r_u=0.3$ 1.3 $r_u=0.2$ 1.4 $r_u=0.15$ 1.5	3	6. END OF CONSTRUCTION PORE PRESSURES IN DAM AND FOUNDATION BEFORE RESERVOIR IMPOUNDING.		1.3	1.3	1	1.3	4	9. COFFERDAM CREST ELEVATION 1,085. END OF CONSTRUCTION PORE PRESSURES IN COFFERDAM AND FOUNDATION.		1.2	1.4	2	1.3	5	



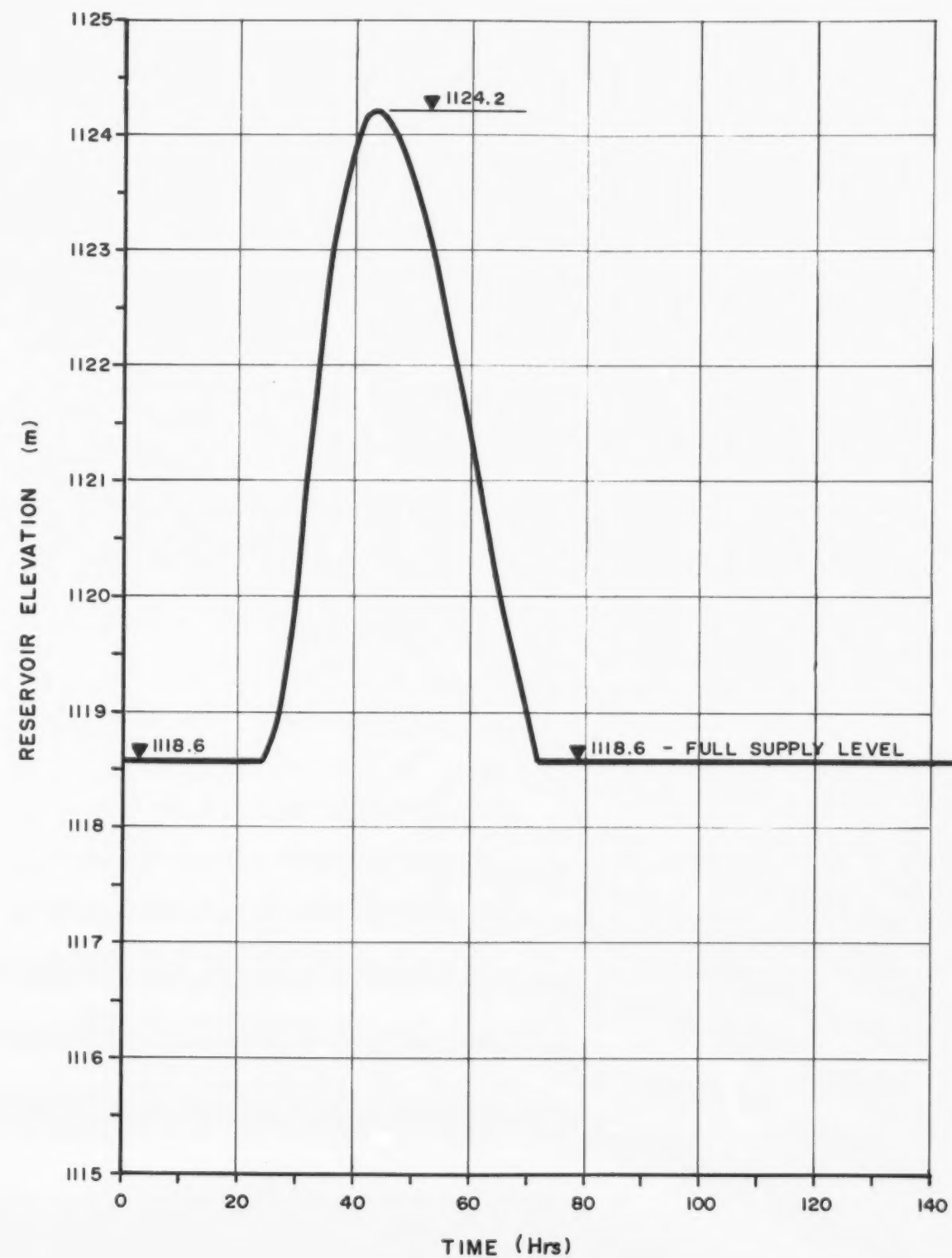
UPSTREAM FAILURE SURFACE	DOWNSTREAM		CASE	MIN. F.S. REQ'D	UPSTREAM		DOWNSTREAM	
	F.S.	FAILURE SURFACE			F.S.	FAILURE SURFACE	F.S.	FAILURE SURFACE
1	1.4	3	10. COFFERDAM CREST ELEVATION 1,075, RIVER LEVEL ELEVATION 1,073, WITH STEADY STATE SEEPAGE CONDITIONS IN DAM AND FOUNDATION.	1.4	2.1	2	1.6	6
2	1.6	5	11. COFFERDAM CREST ELEVATION 1,075 AND CONSTRUCTION PORE PRESSURES IN DAM AND FOUNDATION.	1.2	1.4	2	1.3	6
2	1.3	5						

MATERIAL	UNIT WEIGHT γ (kN/m ³)	COHESION c' (kPa)	ANGLE OF APPARENT SHEARING RESISTANCE ϕ' (DEGREES)	PORE PRESSURE COEFFICIENT γ_u
A - COMPACTED TILL	21.6	0	24	0.4
B - GRANULAR FILL	21.6	0	35	0
C - RANDOM FILL	21.6	0	35	0.1
D - RIVER ALLUVIUM	21.6	0	35	0.1
E - RIVER COLLUVIUM	21.6	0	25	0.3
F - CLAYSTONE (CROSS BEDDING)	23.6	0	40	0.3
G - SANDSTONE (CROSS BEDDING)	23.6	0	55	0.2
H - WEAK SEAMS IN CLAYSTONE	23.6	0	17.5	0.3
J - WEAK SEAMS IN MUDROCK STRUCTURAL UNIT No. 1.	23.6	0	12	0.3

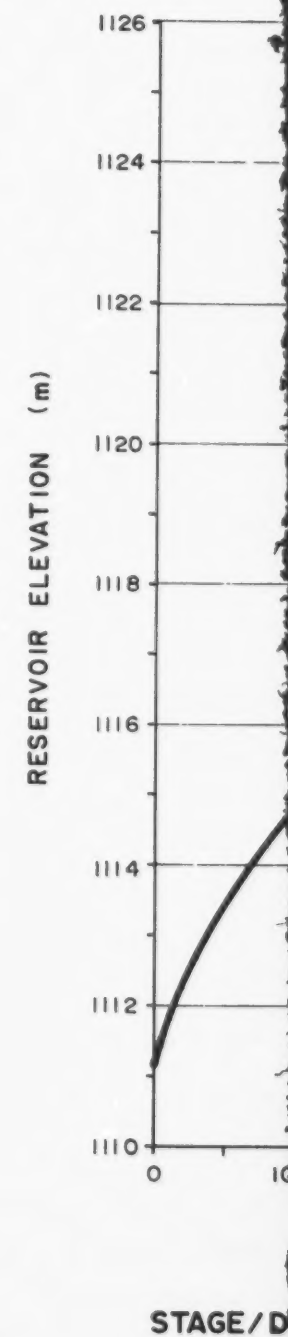


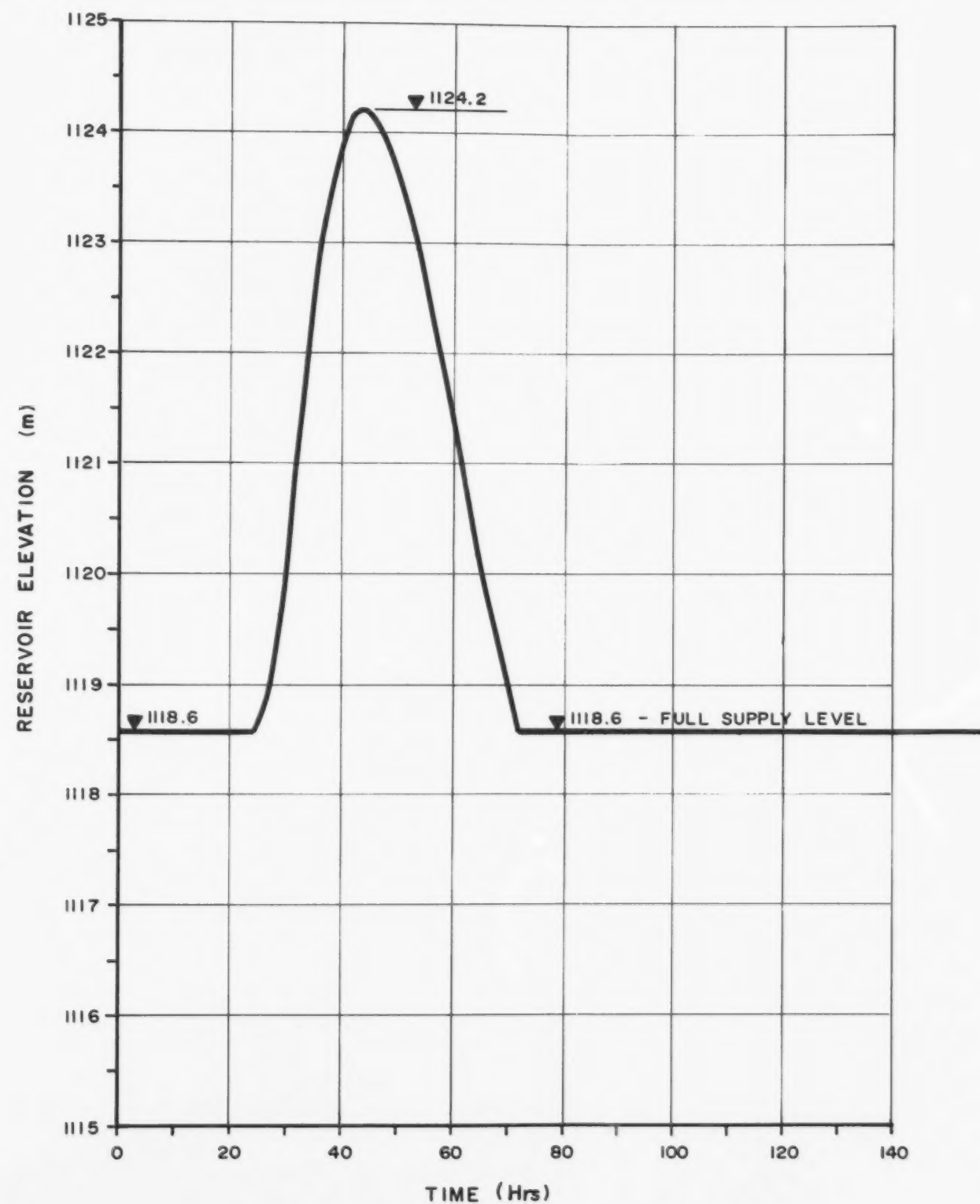


PROBABLE MAXIMUM FLOOD - INFLOW & OUTFLOW
HYDROGRAPHS

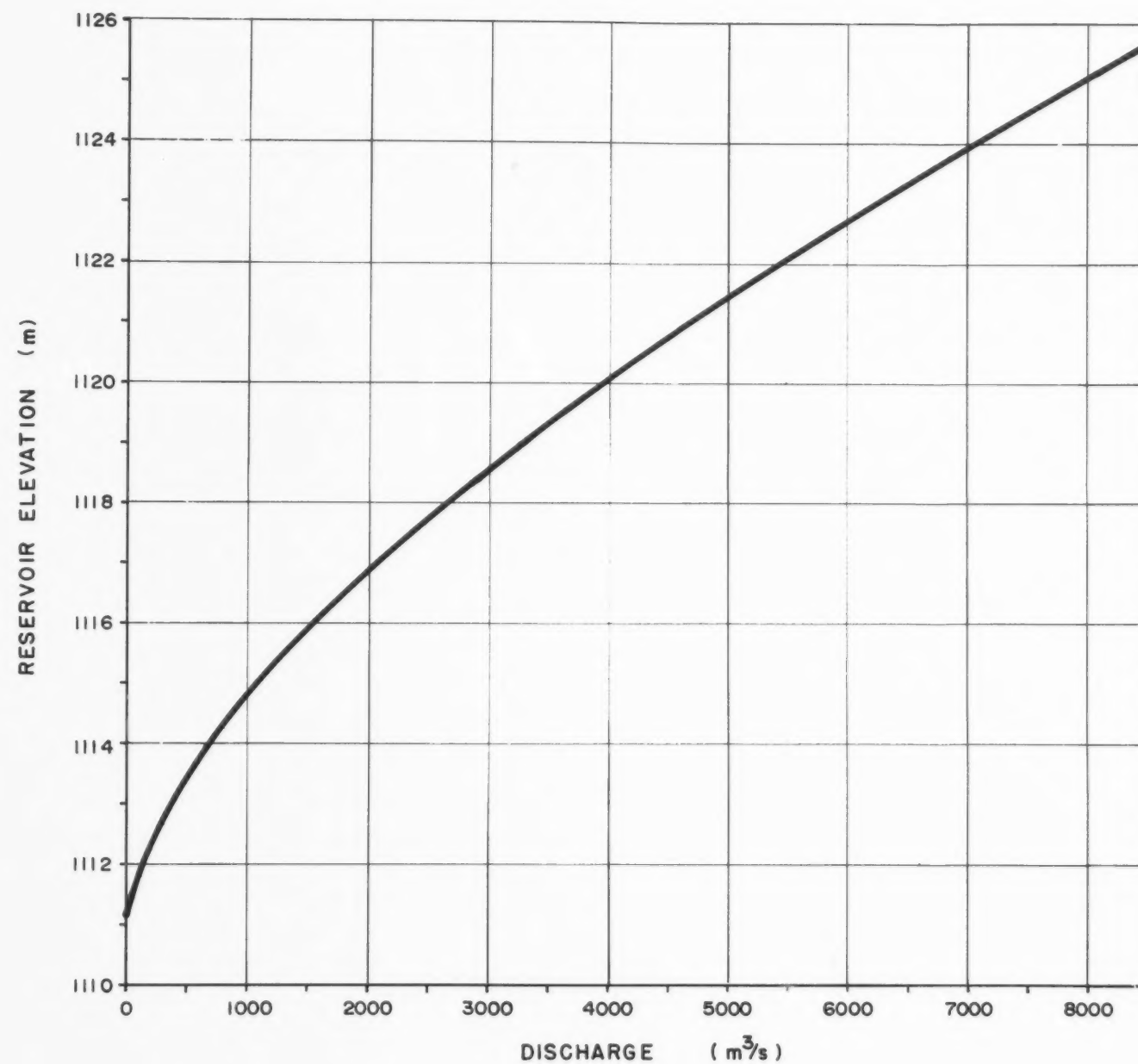


PROBABLE MAXIMUM FLOOD - RESERVOIR SURCHARGING

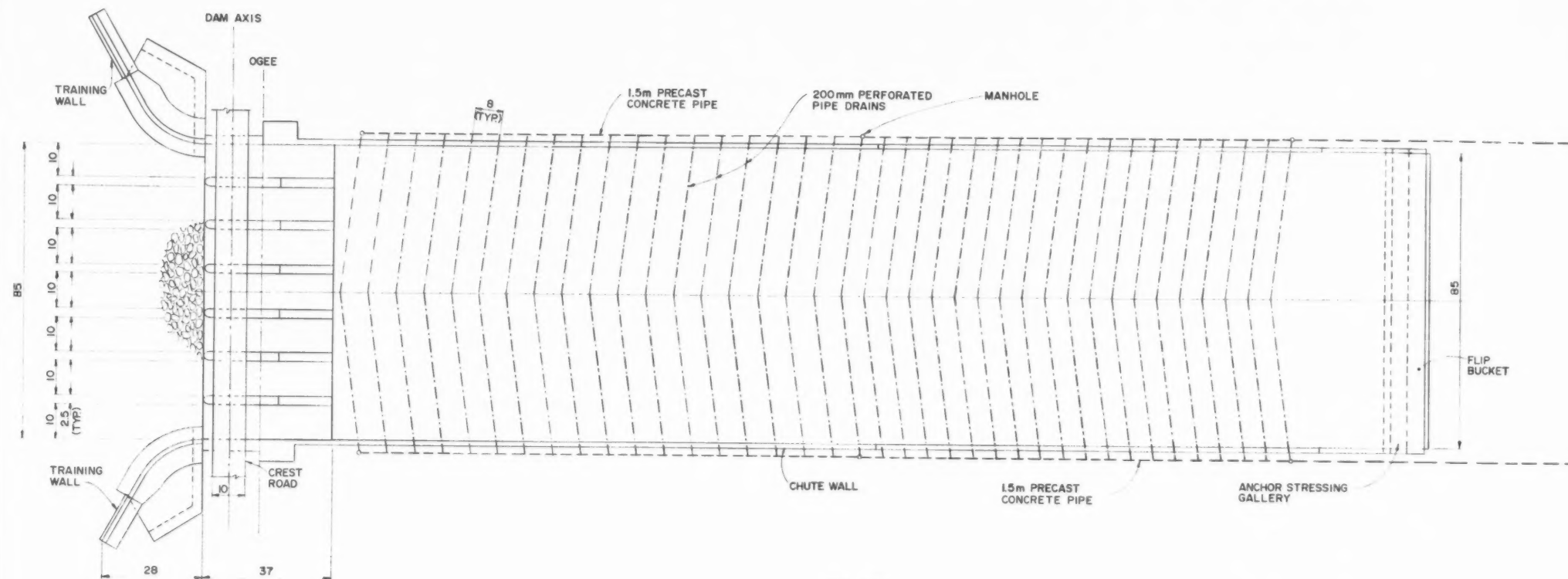




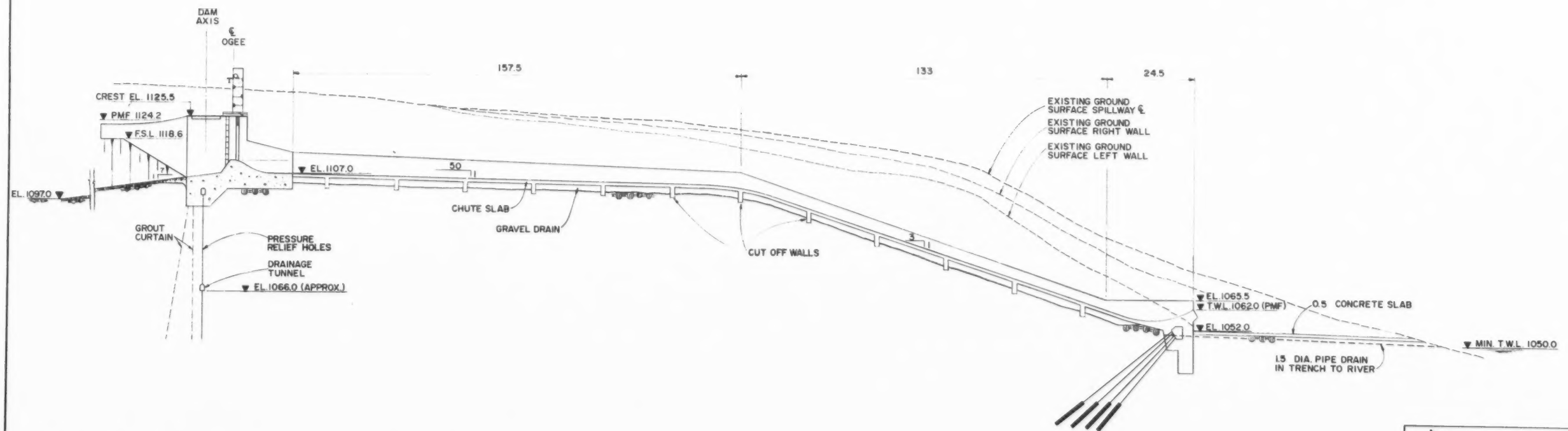
PROBABLE MAXIMUM FLOOD - RESERVOIR SURCHARGING



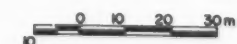
STAGE/DISCHARGE CURVE - GATES FULL OPEN

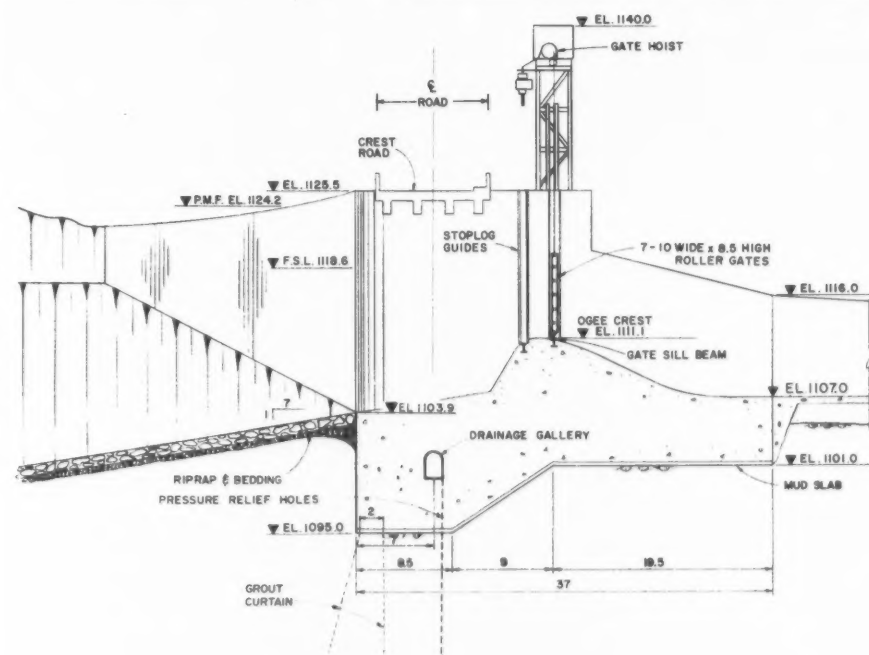


PLAN

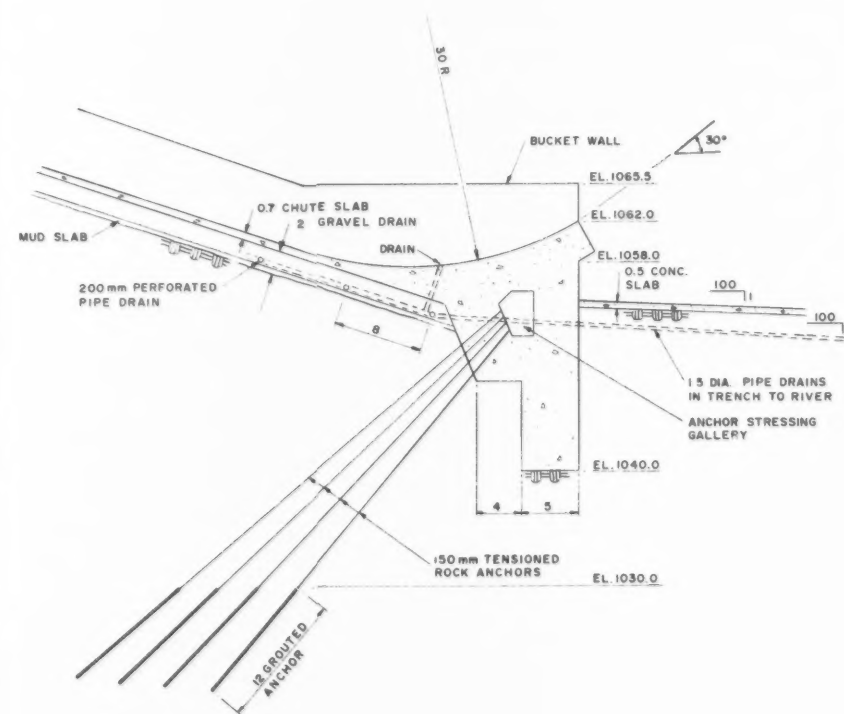


SECTION

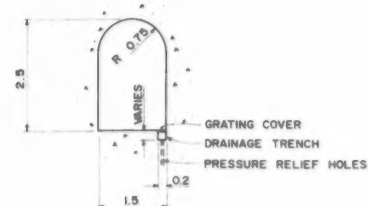




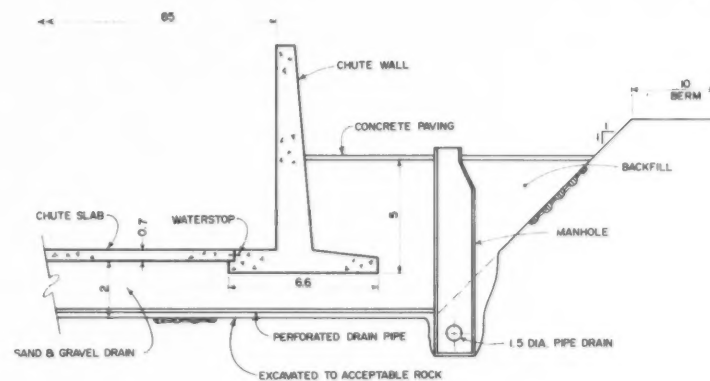
HEADWORKS



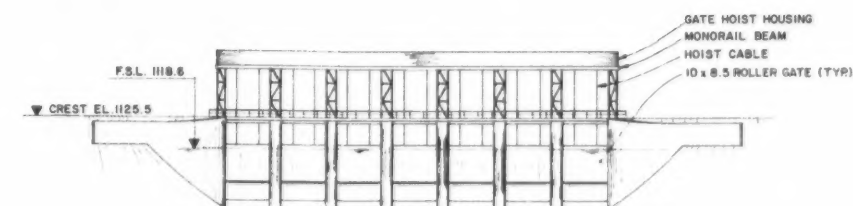
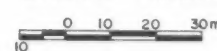
FLIP BUCKET



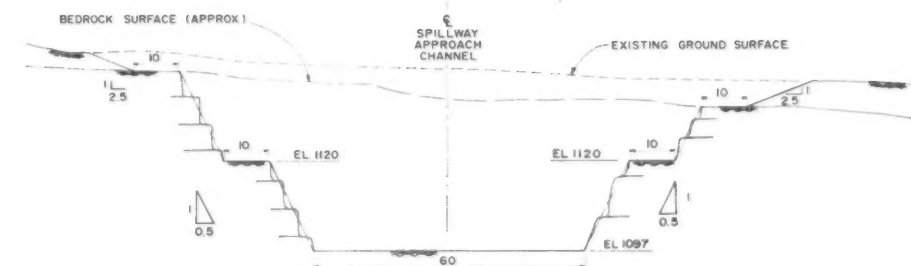
DRAINAGE GALLERY



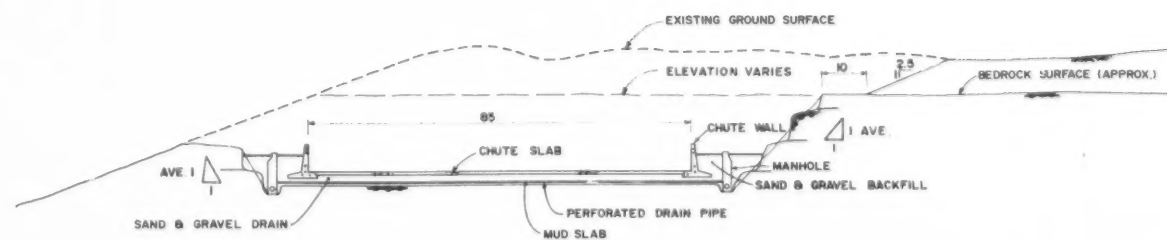
CHUTE WALL



UPSTREAM ELEVATION

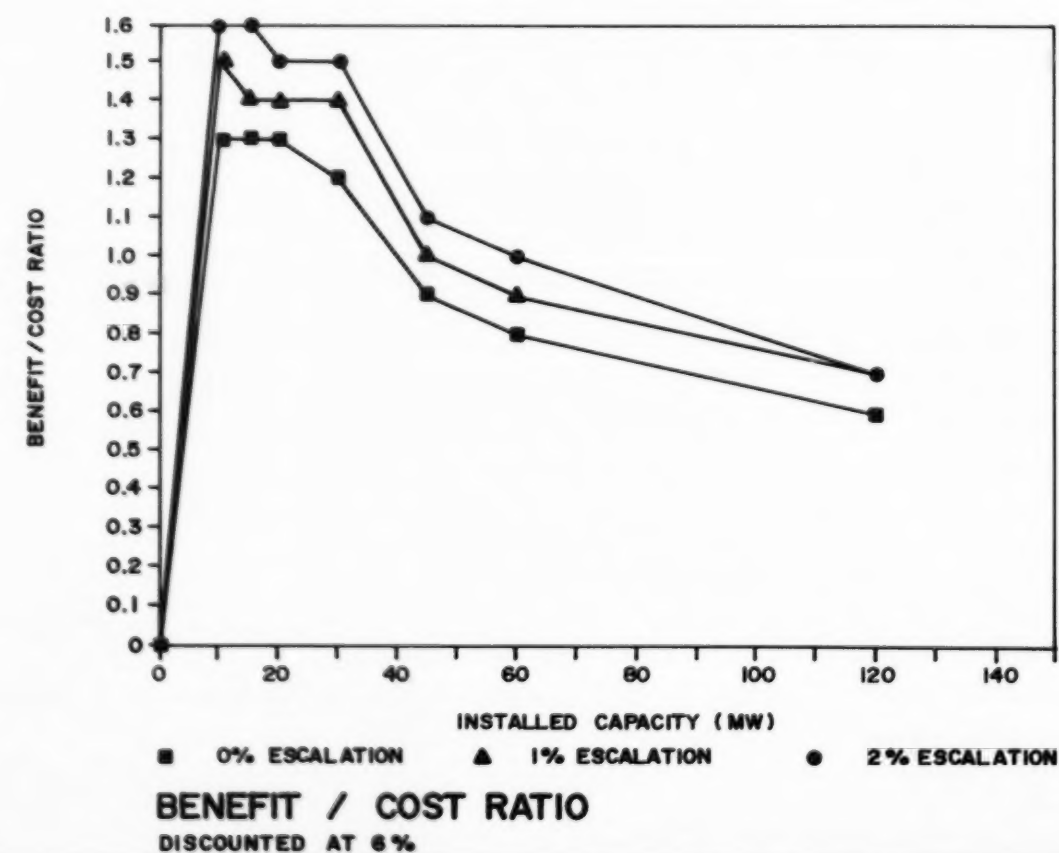
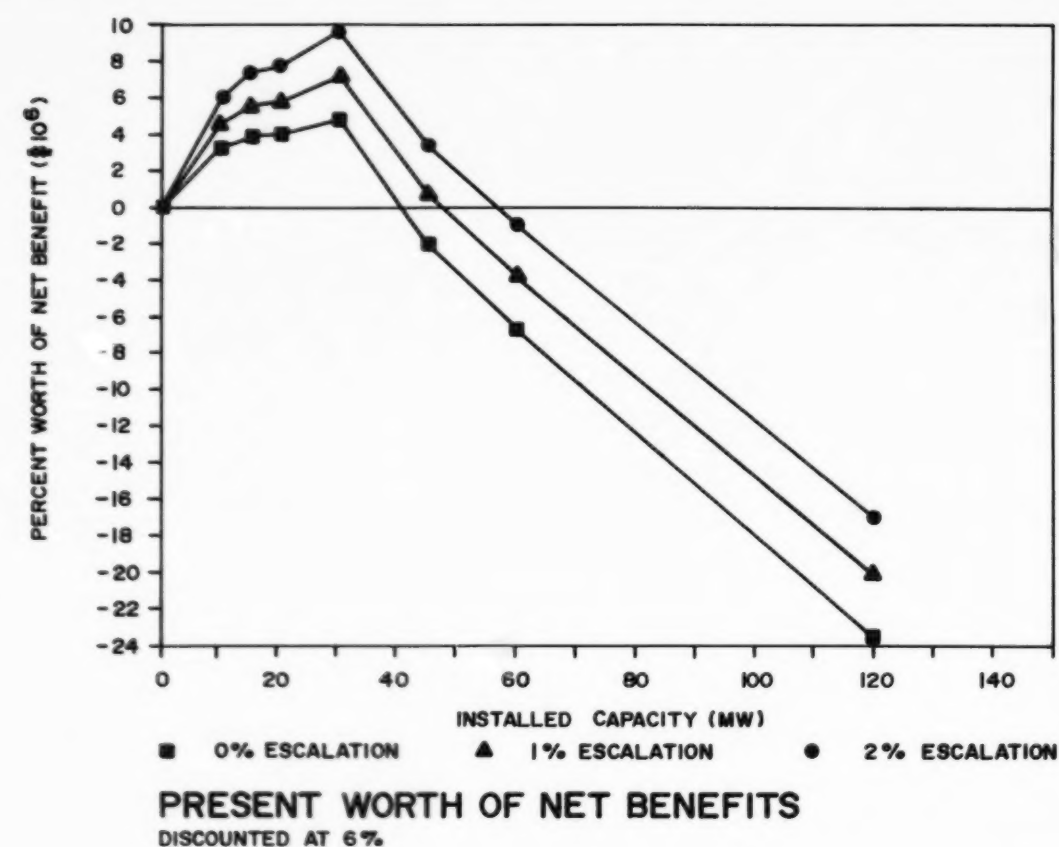
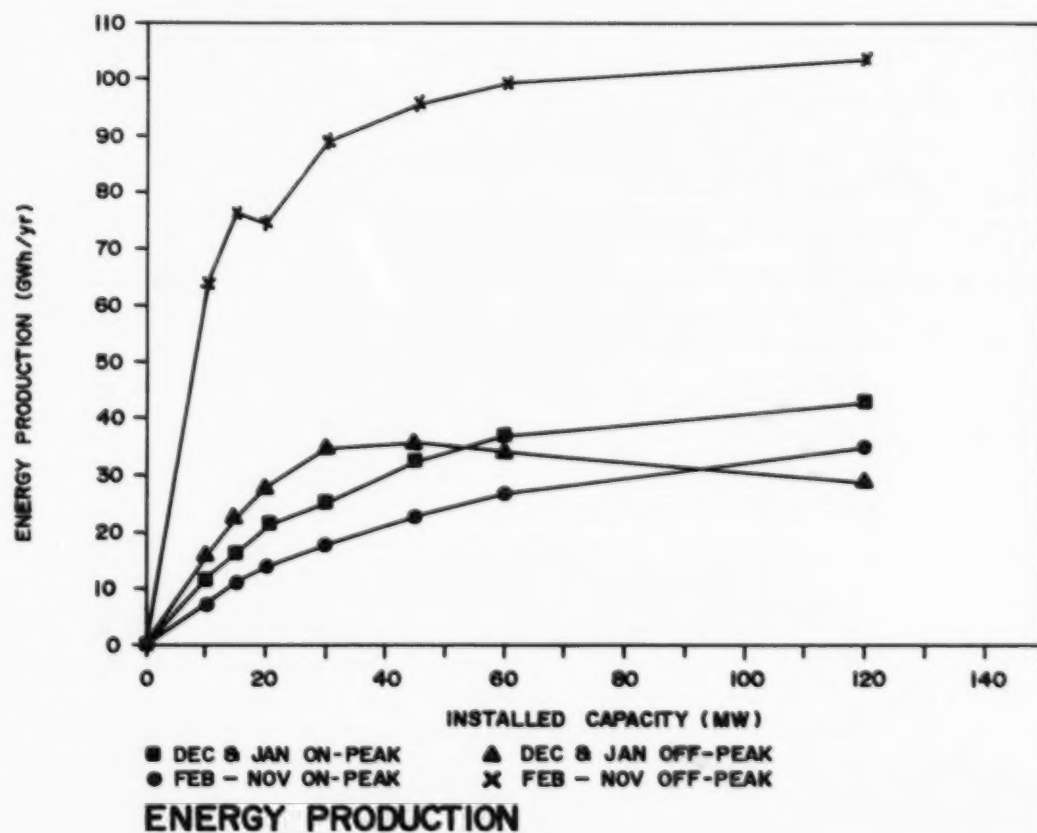
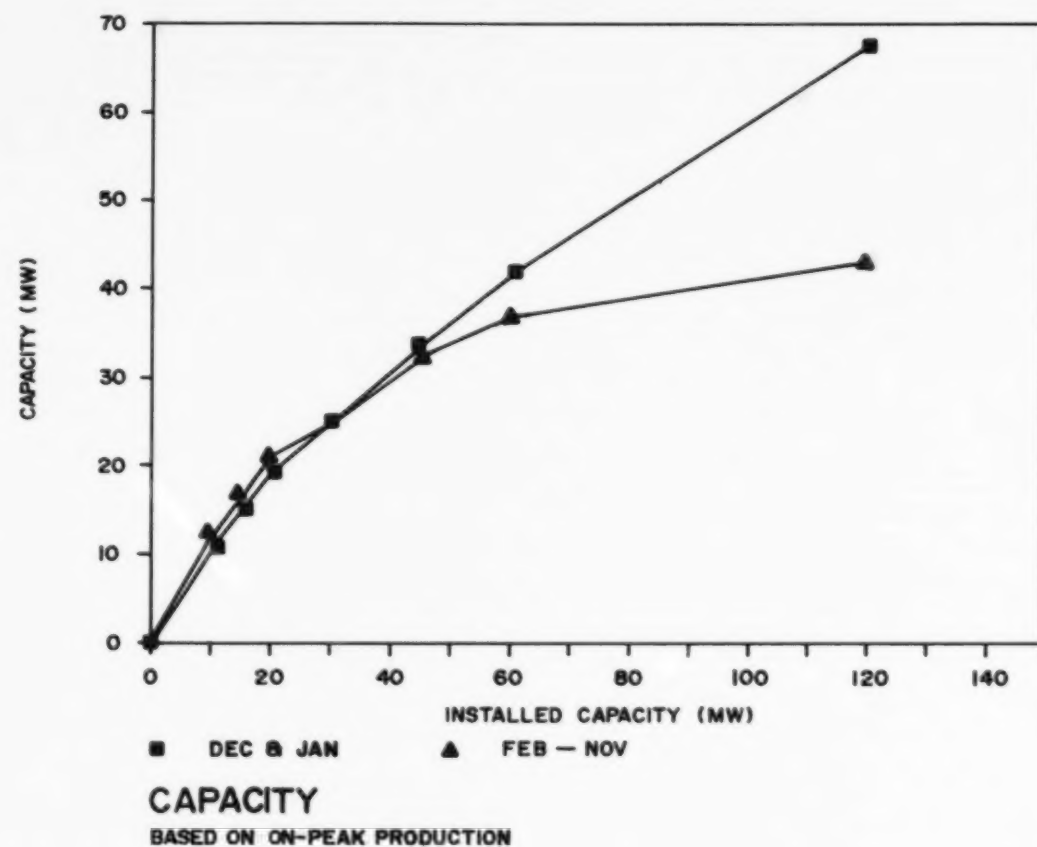


SPILLWAY APPROACH CHANNEL SECTION



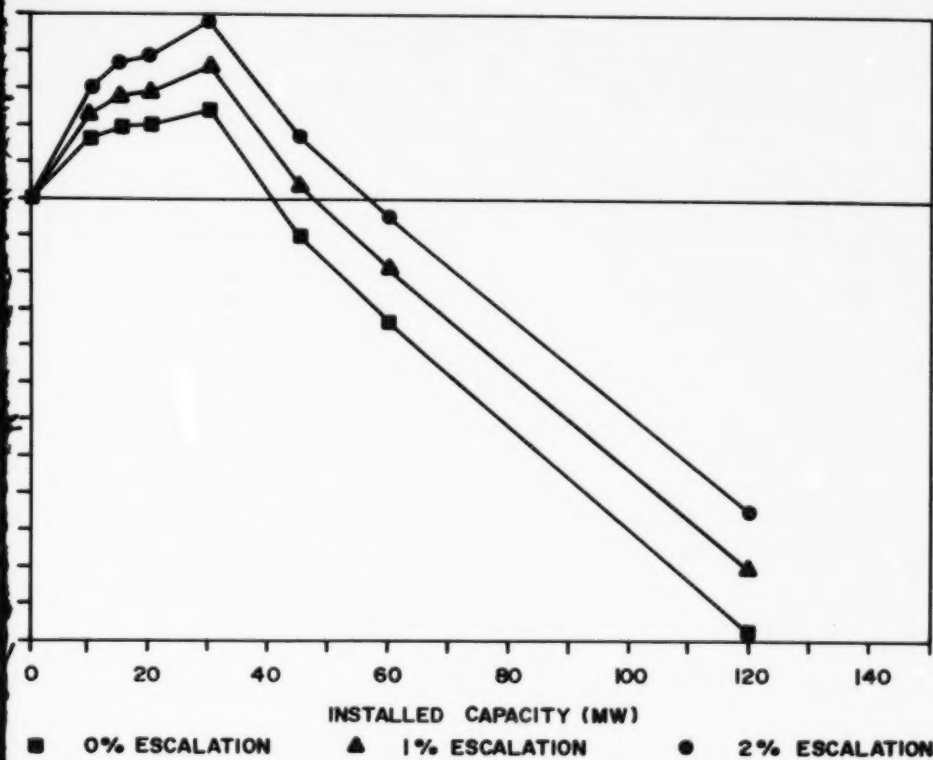
SPILLWAY CHUTE SECTION



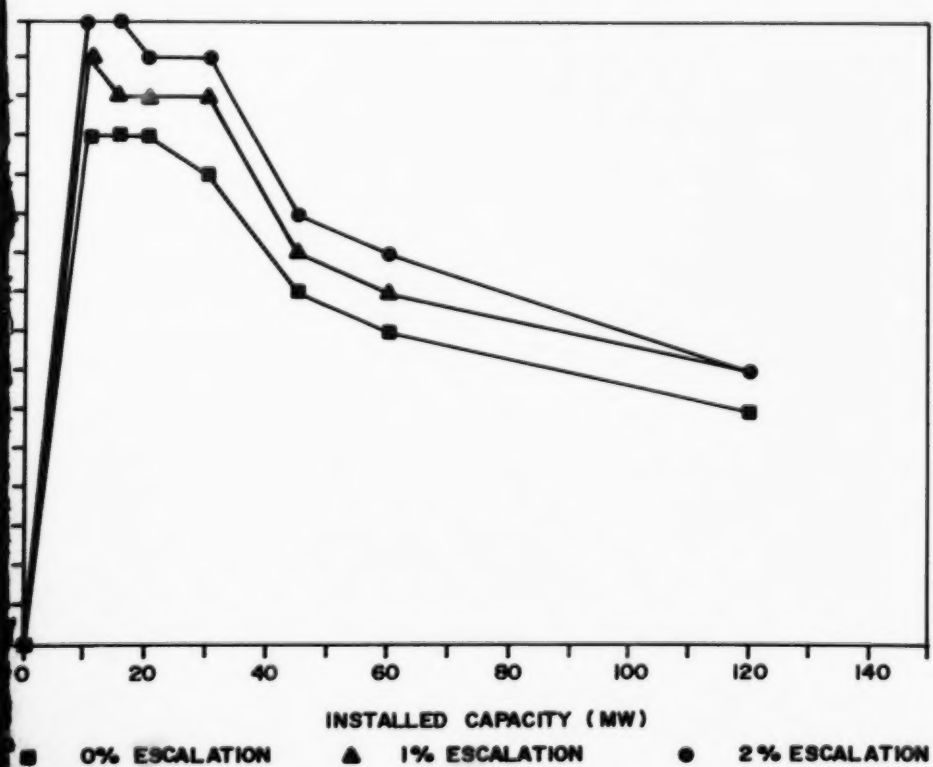


ANNUAL BENEFITS ($\$10^6/\text{yr}$)

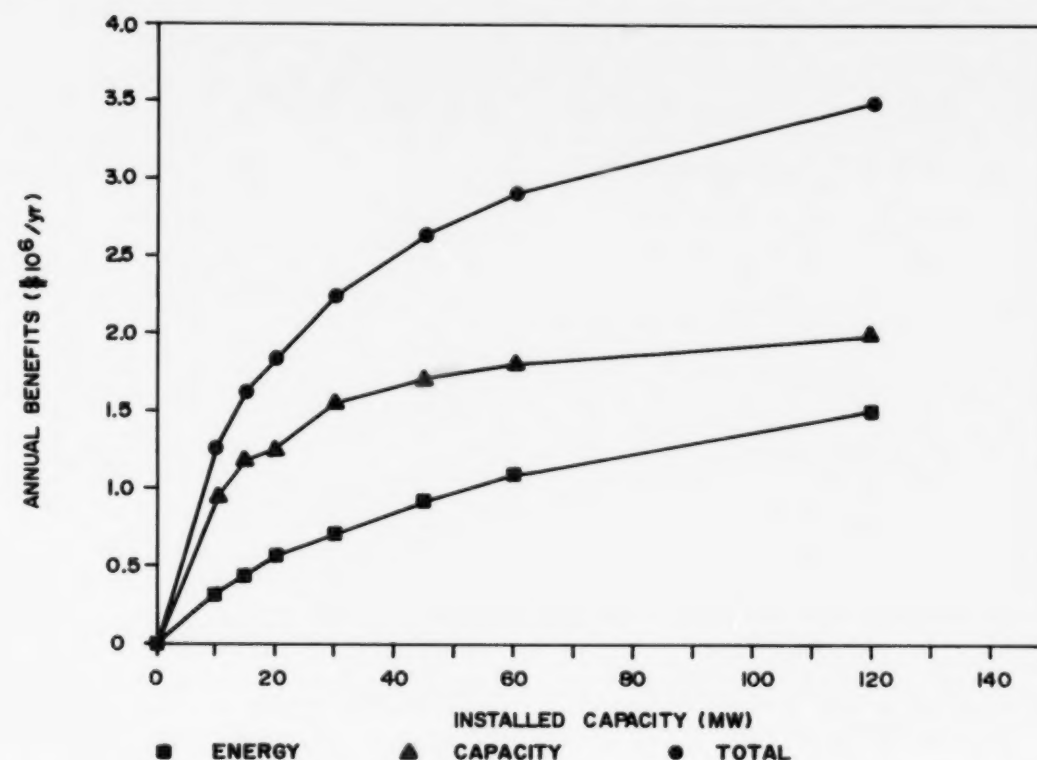
ANNUAL BENEFITS ($\$10^6/\text{yr}$)



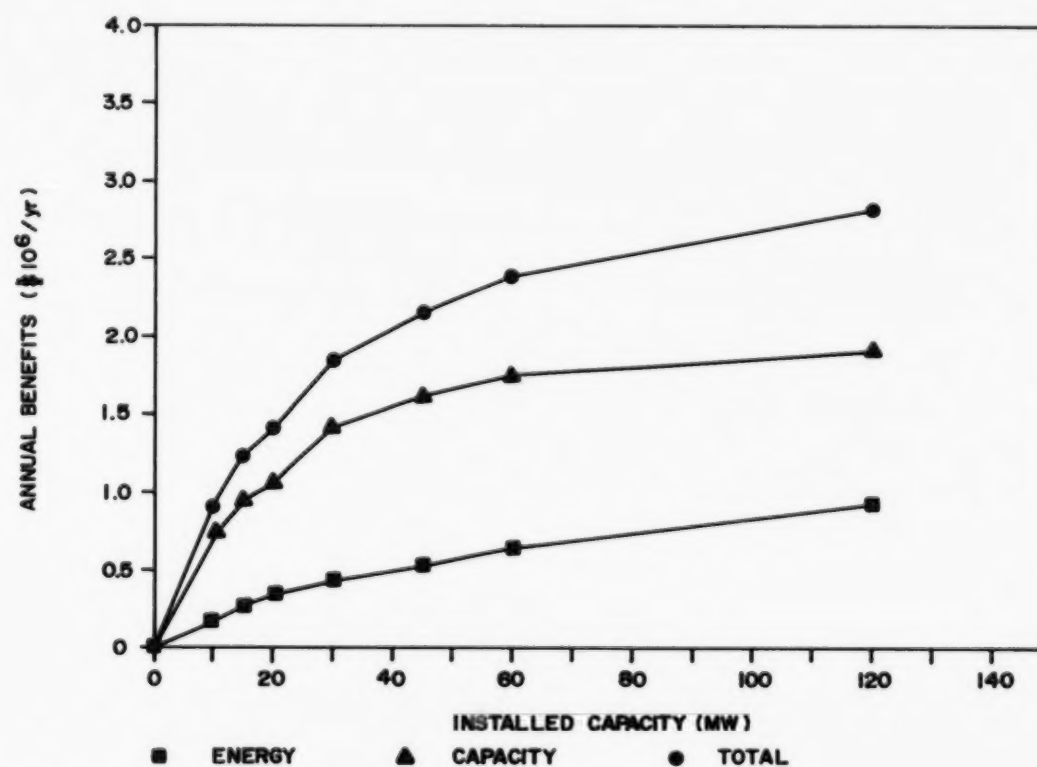
PRESENT WORTH OF NET BENEFITS
DISCOUNTED AT 6%



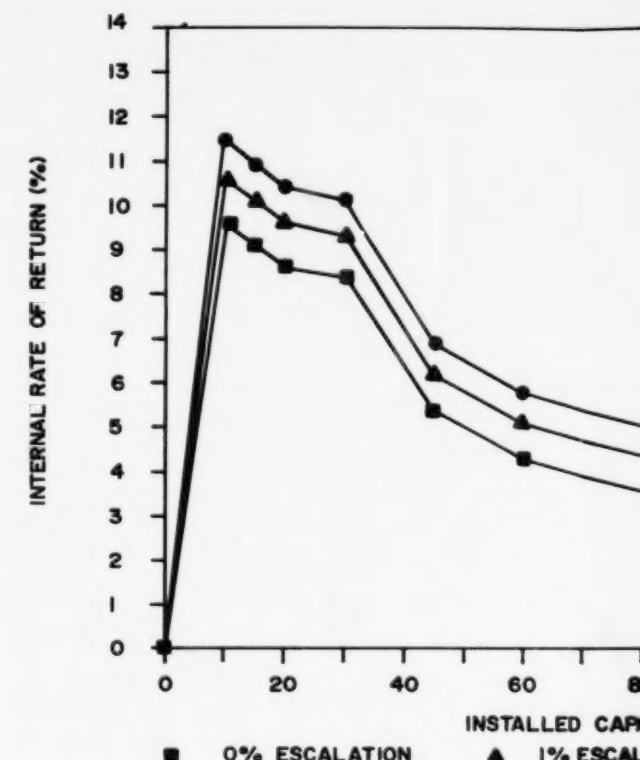
BENEFIT / COST RATIO
DISCOUNTED AT 6%



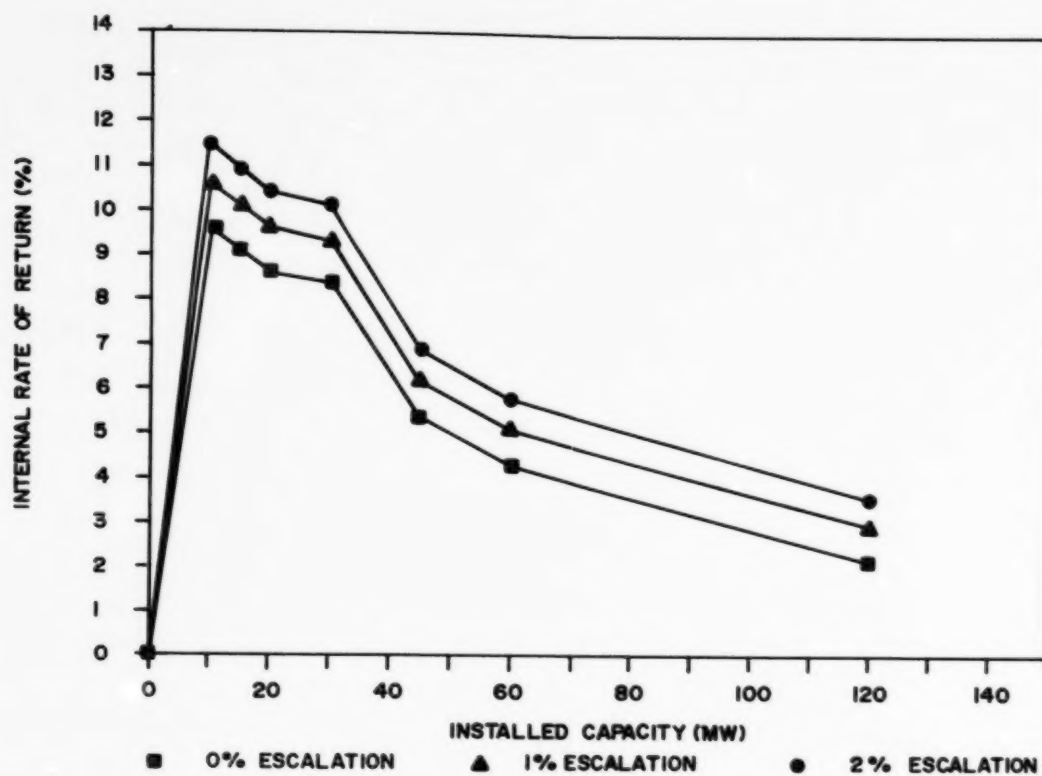
ANNUAL BENEFITS
DEDICATED STORAGE



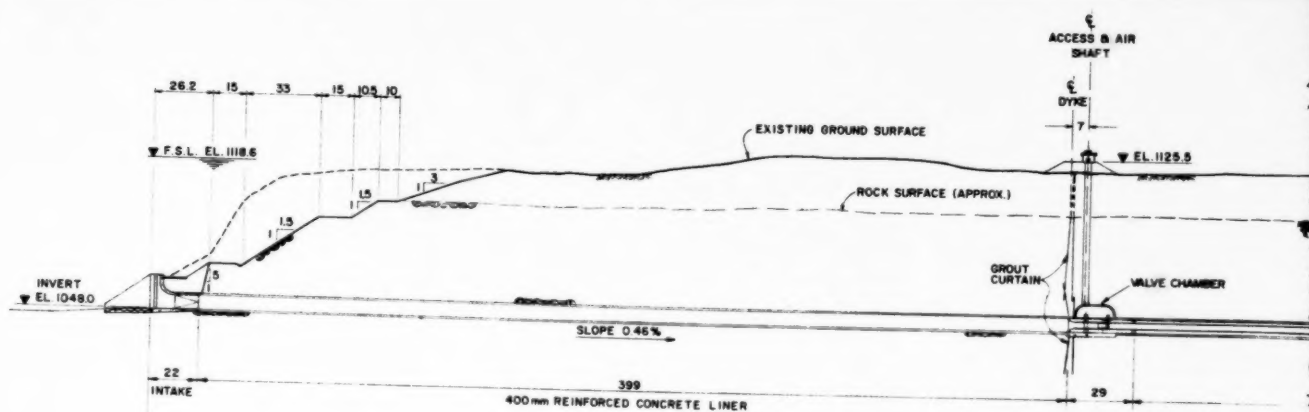
ANNUAL BENEFITS
MATURE SYSTEM



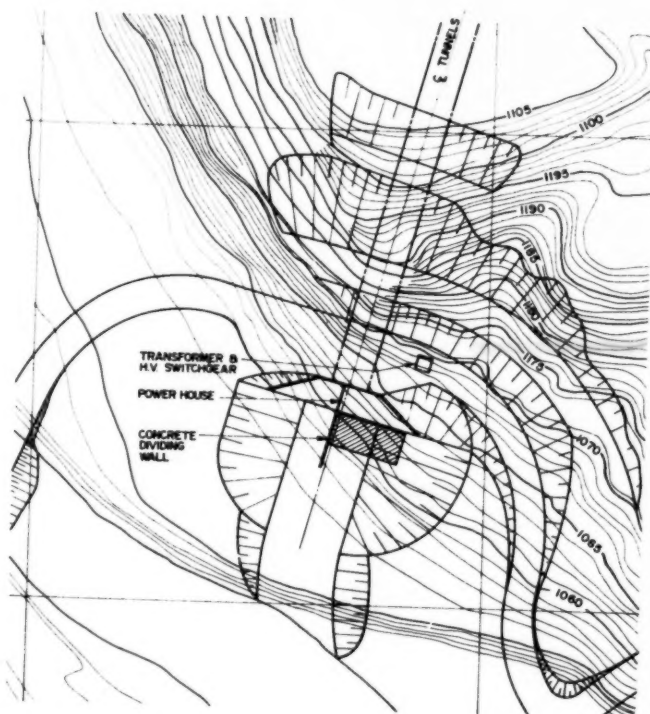
INTERNAL RATE OF RETURN



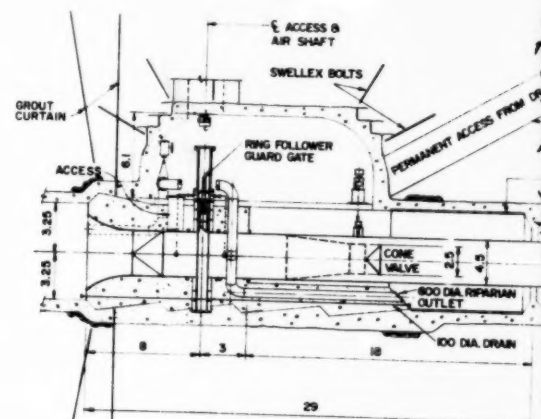
INTERNAL RATE OF RETURN



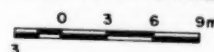
LONGITUDINAL SECTION



SITE PLAN

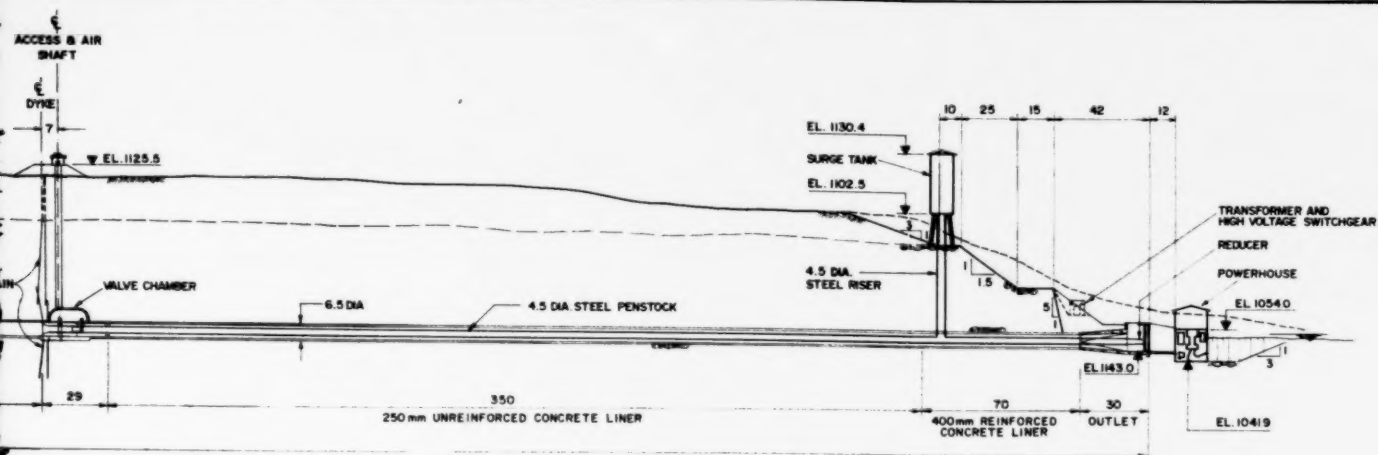


VALVE CHAMBER



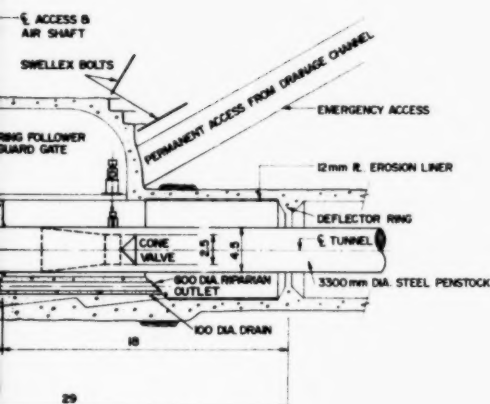
CONFIGURATION	A	B	C	PI. LOCK HYD. 1175
1 - 10 MW	11.5	13.5	12.2	2.7
1 - 15 MW	12.5	14.0	12.6	3.3
1 - 20 MW	15.6	17.1	12.8	3.8
2 - 15 MW	22.0	14.0	12.8	4.8

POWERHOUSE SIZES



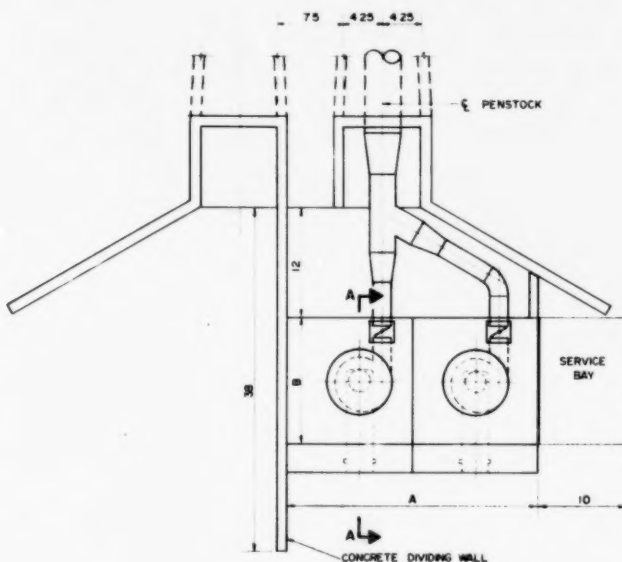
LONGITUDINAL SECTION

0 40 60m



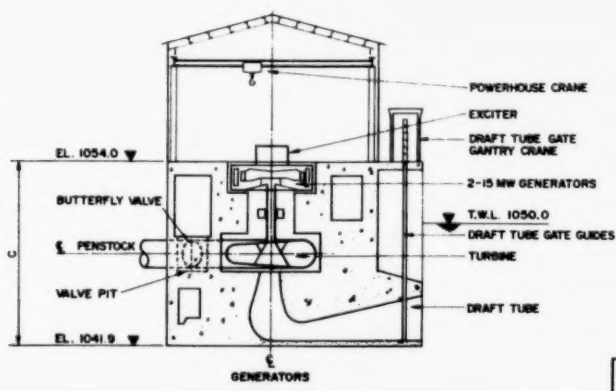
VALVE CHAMBER

6 9m



POWERHOUSE PLAN AT OUTLET PORTAL

0 5 10 15m



SECTION A-A

POWERHOUSE SECTION

0 3 6 9m

POWERHOUSE SIZES

A	B	C	PENSTOCK DIAMETER
11.5	15.5	12.2	2.7
12.5	14.0	12.5	3.3
13.5	17.1	12.5	3.8
14.0	14.0	12.5	4.5

OLDMAN RIVER DAM

POWER FACILITIES

PLATE 34